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December 1998

US Army Corps of Engineers
Waterways Experiment Station

Coastal Inlets Research Program

Shinnecock Inlet, New York, Site Investigation

Report 2
Evaluation of Sand Bypass Options

by *Gregory L. Williams, Andrew Morang, Linda Lillycrop*

Prepared for Headquarters, U.S. Army Corps of Engineers
Under Inlet Geomorphology and Channel Evolution
Work Unit 32930

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U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

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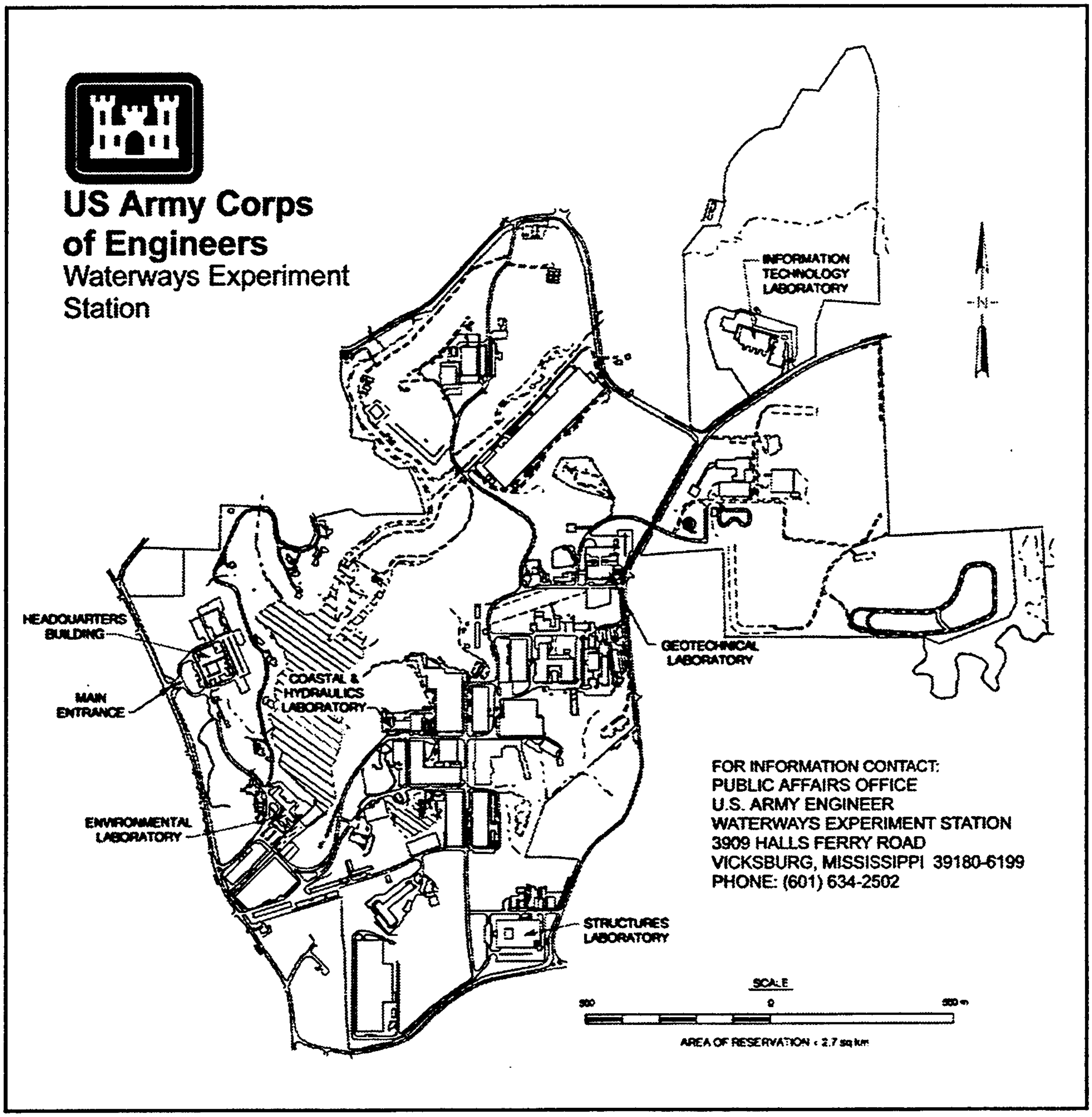

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Preface

This study was performed by the U.S. Army Engineer Waterways Experiment Station (WES), Coastal and Hydraulics Laboratory (CHL), Coastal Sediments and Engineering Division (CSED), Coastal Evaluation and Design Branch (CEDB). Funding for the study was provided by the U.S. Army Engineer District, New York, through the Fire Island to Montauk Point Reformulation Study. Additional technical review and minor updates were supported by the Inlet Geomorphology and Channel Evolution Work Unit 32930, Coastal Inlets Research Program (CIRP). Mr. Edward B. Hands (CEDB) was the CIRP Principal Investigator. Dr. Nicholas C. Kraus was the Technical Manager, and Mr. E. Clark McNair was the CHL Program Manager for CIRP. Messrs. John Bianco, Charles Chesnutt, and Barry W. Holliday were Headquarters, U.S. Army Corps of Engineers Program Monitors.

Work was performed under the general supervisory direction of Dr. Yen-hsi Chu, Chief, Engineering Applications Unit, CEDB; Ms. Joan Pope, Chief, CEDB; Mr. Thomas Richardson, Chief, CSED; Mr. Charles C. Calhoun, Jr., Assistant Director, CHL; and Dr. James R. Houston, Director, CHL.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. The Commander was COL Robin R. Cababa, EN.

Conversion Factors, Non-SI to SI Units of Measurements

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic yards	0.7645549	cubic meters
feet	0.3048	meters
square feet	0.00929	square meters
miles (U.S. statute)	1.609347	kilometers
knots	0.514	meters per second
inches	25.4	millimeters
^{0}F	subtract 32, then multiply by 5/9	^{0}C
horsepower	0.7457	kilowatts
feet of water (pressure)	304.8	liq/sq meter
tons	0.9078	metric tons

1 Introduction

Shinnecock Inlet is the easternmost of six openings in the barrier island chain along the south shore of Long Island, New York (Figure 1). The barrier island chain encloses a series of coastal bays and tidal marshes. Shinnecock Inlet is located in the town of Southampton, 153 km[1] by sea east of the Battery, New York, and 60 km southwest of Montauk Point. The inlet is stabilized by two rubble-mound jetties constructed in the mid-1950s (Figure 2). A Federal navigation channel connects Shinnecock Bay to the Atlantic Ocean, through which boaters can access the Long Island Intracoastal Waterway. Shinnecock Bay, an irregularly shaped body 14.5 km long (east-west) and 0.6 to 4.5 km wide with water depths mostly less than 10 m, is connected by channels to Moriches Bay to the west and by the Shinnecock Canal to Peconic Bay on the north. Several small creeks drain into the northern side of the bay, including Penniman, Stone, Phillips, and Weesuck. These creeks do not provide much freshwater input. The total water surface area of Shinnecock Bay is about 4,100 hectares (10,240 acres). A fish-handling facility is located just west of the inlet on the north side of the barrier. The commercial fishing fleet depends upon Shinnecock Inlet for access to offshore fishing grounds because no convenient alternate access exists.

There are several ongoing shore protection studies being conducted by the U.S. Army Engineer District, New York (NAN), along the south shore of Long Island. Shinnecock Inlet falls within the largest effort, the "Fire Island to Montauk Point Reformulation Study" (FIMPRS), which is examining coastal processes, shore protection, and flood damage reduction alternatives from Fire Island Inlet eastward to Montauk Point. One part of the FIMPRS is an evaluation of inlet sand management alternatives at Shinnecock Inlet to address the interruption of regional longshore transport. Since the jetties were constructed in 1952 and 1953, the beach east of Shinnecock Inlet has accreted, while the area extending 600 m west of the inlet has eroded. This report discusses the geomorphic history of the inlet and, using the results of a coastal processes

[1]Units of measurement in the text of this report are shown in SI units, occasionally followed by non-SI (British) units in parentheses. Maps have been plotted in state plane coordinate system in feet to be consistent with charts normally used by U.S. Army Engineer District (USAED), New York. A table of factors for converting non-SI units of measurement to SI units is presented on page x.

study conducted by Moffatt & Nichol Engineers (MNE) under contract to NAN, evaluates ebb shoal morphology and longshore transport processes as they relate to sand management (bypass) options. Five bypass options are evaluated based on cost, operational effectiveness, and sand source location. A decision matrix is included to compare alternatives.

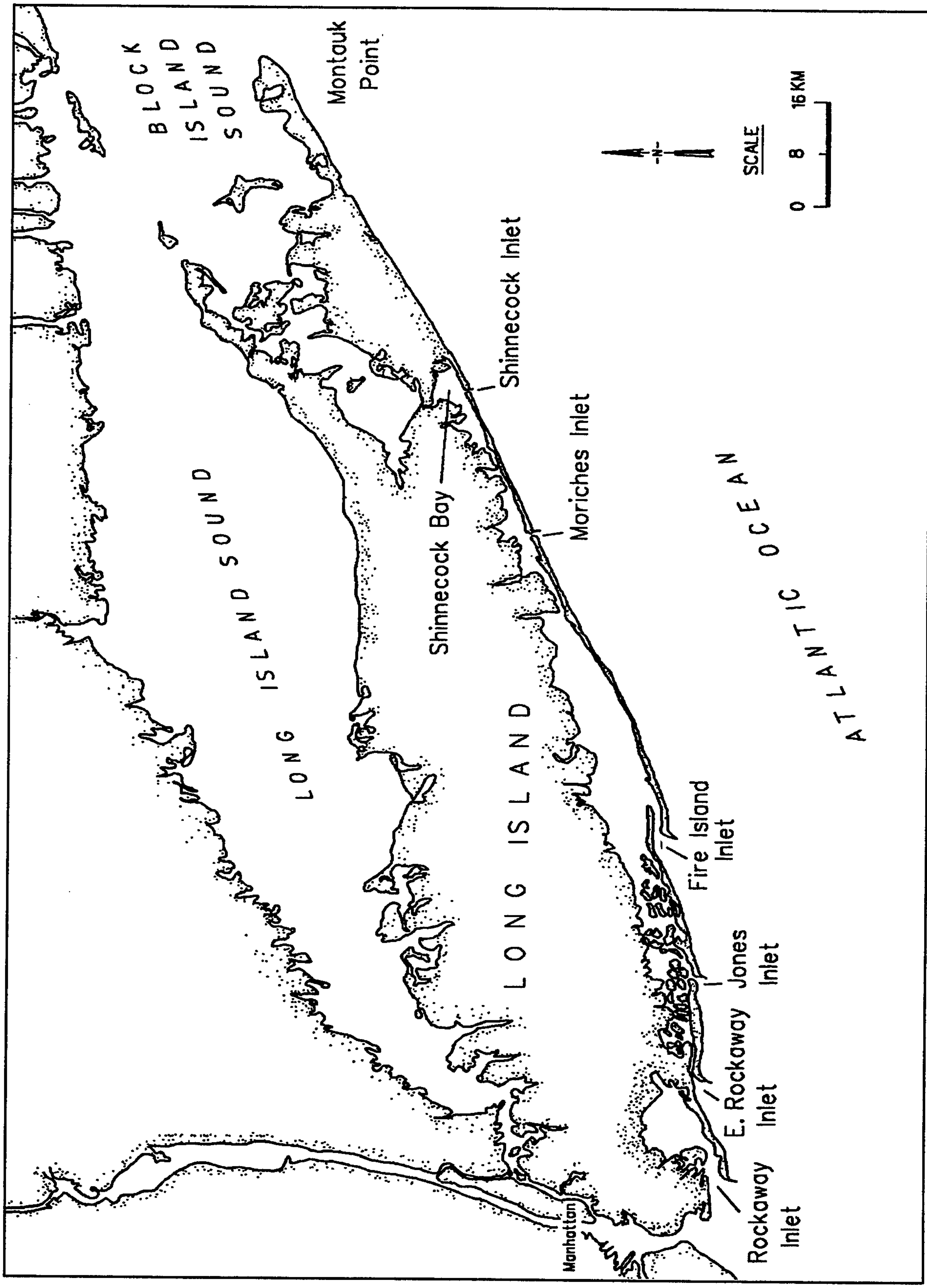

Figure 1. Long Island, New York, location map

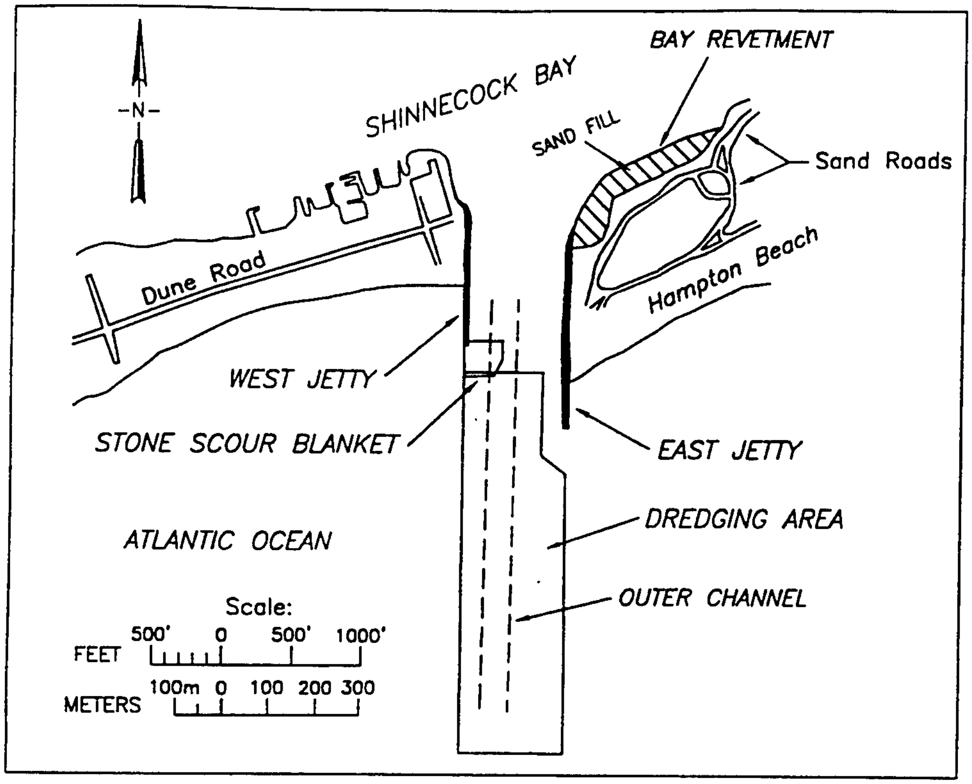

Figure 2. Shinnecock Inlet and Federal Navigation Project

2 Geologic Setting and Morphologic Development

Geologic Setting

Long Island is the largest island adjoining the continental United States. It is 190 km long and extends from the Narrows at the entrance to New York Harbor eastward to Montauk Point, due south of the Connecticut-Rhode Island boundary. The island is part of the Atlantic Coastal Plain, with basement of Cretaceous age rock and some older metamorphic rocks that outcrop in the extreme west near Long Island City. Coastal plain deposits are exposed only in the western part of the island. Most of both the surficial and the underlying materials are Pleistocene morainal and outwash accumulations associated with continental glaciers (Fuller 1914).

Two morainal ridges run the length of Long Island, with the southern one, the Ronkonkoma, extending to Montauk Point. Most of the north shore of the island facing Long Island Sound consists of bluffs 10 to 30 m high. South of the southern ridge, an outwash plain of fine gravel and sand stretches southward for 1 to 15 km to the Atlantic Ocean (Fuller 1914). Off the south shore, a more or less continuous barrier encloses broad, shallow Jamaica, Hempstead, Great South, Moriches, and Shinnecock Bays. Coney Island, once the westernmost extension of the barrier chain, is now artificially attached to the mainland. At present, six inlets provide access to the bays (Table 1).

The barrier ends at Southampton, and from there to Montauk Point, the coast follows a nearly straight line intersecting old headlands and crossing old bays. One of the bays, Mecox, is occasionally open to the Atlantic via an intermittent inlet. The exposed bluffs along this eastern portion of the south coast are generally considered to be the source of sediment that feeds the development of the barrier beaches to the west. The direction of longshore drift is predominantly westward along the entire south shore, but local reversals occur near the inlets. Although the dominant westward drift has been recognized for decades, McCormick and Toscano (1981), Williams and Meisberger (1987), and even Fuller (1914) proposed that some sediment may be moving onshore from the shelf to augment that moved by longshore currents. Almost no sand is delivered to the coast by streams.

Table 1 Inlets Along the South Shore of Long Island				
Inlet	**Bay or Sound**	**Island or Beach to West**	**Island or Beach to East**	**Distance from West Tip of Coney Island (km)**
Rockaway	Jamaica	Coney Island	Rockaway Beach	5.5
East Rockaway	Hempstead via Reynolds Channel	Rockaway Beach and City of Far Rockaway	Atlantic Beach (west end of Long Beach Island)	23
Jones	Hempstead	Point Lookout (east end of Long Beach Island)	Short Beach (west end of Jones Beach)	38
Fire Island	Great South	Cedar Island Beach (east end of Jones Beach)	Robert Moses State Park, Fire Island	61
Moriches	Moriches	Fire Island	Westhampton Beach	111.5
Shinnecock	Shinnecock	Tiana Beach	Southampton Beach	136

Beach sand is primarily quartz and feldspar, although storm lag deposits of magnetite and garnetiferous (heavy mineral) sands are often found on the beach face after storms. Along the barrier shore, gravel is generally scarce, but accumulations are sometimes seen where the beaches connect with the mainland, such as near Westhampton and Southampton (Fuller 1914).

The marshes along the south coast are rich habitats for numerous species of birds and fish and support the productive growth of marsh grasses. The most common salt-marsh species include black grass (*Juncus gerardi*), various salt-marsh types, (*Spartina patens* association), salt thatch (*Spartina glabra* Muhl.), and Eel grass (*Zostera marina* L.) (Fuller 1914). Submerged tree stumps and peat beds in various parts of Long Island, indicators of a relative sea level rise, have been described by many writers (e.g., Rampino and Sanders 1980).

The beaches near Shinnecock Inlet provide nesting habitat for a number of bird species. The common tern (*Sterna hirundo*) and the least tern (*Sterno albifrons*) are of particular importance. In addition, the roseate tern (*Sterna dougallii*) has been sighted in the project area on Warner Islands (USAE District, New York 1988). Another colonial shore bird, the black skinner (*Rynchops niger*) also nests in the project area, as does the noncolonial piping plover (*Charadrius melodus*). Nesting habitat for all of these species must be preserved, and thus may place constraints on the times of the year when beach renourishment and other work can proceed.

Barrier Island Migration and Sea Level Change

One of the factors that affects shoreline position on sandy coasts is the rise or fall of relative sea level (rsl). In this section, we summarize findings that the Long Island barriers have retreated for thousands of years and evaluate the evidence that rsl is still rising in this area.

Along the northeast United States, sea level has risen about 90 to 100 m since the end of the Pleistocene epoch, about 12,000 to 15,000 years ago (Nummedal 1983). This Holocene transgression flooded the continental shelves and caused the retreat of barrier islands along much of the eastern seaboard. How do barriers respond to a marine transgression? Two contrasting hypotheses have been proposed: One states that as the sea rises, barriers migrate continuously landward. During this retreat, the breaker zone traverses the entire area that is submerged. Barrier retreat is most likely to occur along shores where there is a large sediment supply and where the rise in sea level is slow. This form of retreat in response to marine transgression has been documented in Rhode Island, where peat exposed on the ocean shoreface demonstrates how former lagoonal sediments are being unearthed seaward of a retreating shoreline (Dillon 1970).

The second hypothesis suggests that barriers can be drowned in place. As the sea rises, the barrier remains fixed, while the lagoon on its landward side deepens and widens. Eventually, the breaker zone reaches the top of the dunes, the barrier is drowned, and the breakers skip landward a considerable distance to form a new barrier at the landward edge of the former lagoon. Under what circumstances could this "skipping" mechanism occur? A barrier might be drowned if there is limited or decreasing sediment supply. Because of the shallow slope of a typical barrier, a vast sediment supply is needed to accommodate even a minor rise of sea level (this is analogous to breakwater construction: a minor increase in height requires a great quantity of extra rock). Without the copious input of sand, the barrier becomes narrower and narrower and is eventually overtopped. Even with a generous sand supply, a period of exceptionally rapid sea level rise might overwhelm the barrier. In addition, if the barrier is densely vegetated, overwash is impaired, resulting in a steepening of the profile as the sea rises. The barrier is unable to migrate landward and can be drowned in place. Details of these theories and the original papers where they were proposed are reprinted in Schwartz (1973). More discussion on the balance between erosion caused by sea level rise versus accretion dependent on sediment supply is found in Engineeer Manual 1110-2-1810 (Headquarters, U.S. Army Corps of Engineers (HQUSACE) 1995) (pp. 2-26).

Based on examination of cores and seismic records off Fire Island, Sanders and Kumar (1975) proposed the following scenario to describe the Holocene submergence of the barriers off Long Island:

When sea level stood at -24 m (9,000 years ago), a chain of barriers existed about 7 km offshore parallel to the modern shore. As the sea rose, the barriers remained in place until the sea reached -16 m, at which time it inundated the top of the dunes. The surf zone was then free to jump about 5 km landward to form a new shoreline about 2 km seaward of the present barrier line. New barriers formed at the -16-m shoreline, becoming ancestors of the modern south shore barriers. These barriers have migrated continuously landward as sea level rose from -16 m to its present elevation. Rampino and Sanders (1980) believed that the “skipping” mechanism explained why complete barrier sediment sequences have been preserved on the Long Island shelf, but Panageotou, Leatherman, and Dill (1985) have disputed this interpretation.

An important question is whether the Long Island barriers are still retreating. Historical evidence indicates they are. In a mapping project based on charts and aerial photographs, Crowell and Leatherman (1985) measured annual net erosion of 0.3 to 1.2 m along most of the south shore barriers between 1834 and 1979. Accretion occurred in the immediate vicinity of Shinnecock Inlet due to the trapping of sand on the updrift fillet. Table 2 summarizes Crowell's and Leatherman's findings. Their evidence points to accelerated erosion after 1933; presumably, much of this occurred after the inlet opened in 1938. Reports indicate that vast amounts of overwash occurred during the September 1938 hurricane. Unfortunately, there is not enough evidence to determine if this one event might have caused a majority of the post-1933 erosion.

Table 2
Shoreline Changes Near Shinnecock Inlet

Period	Zone 3,000 m west of inlet (average)	Near inlet	Zone 3,000 m east of inlet (average)
1834/1838 - 1873/1892	Variable: 1.2-m accretion west of Ponquogue Pt; 0.6- to 0.9-m erosion east of Ponquogue Pt.	Variable	0.6- to 1.2-m retreat
1873/1892 - 1933	0.6- to 0.9-m accretion	0.6- to 0.9-m accretion	0.3- to 1.5-m accretion
1933 - 1979	1.2- to 2.4-m retreat	3.0-m accretion (updrift fillet)	0.3- to 0.9-m retreat
Annual average change 1834/1838 - 1979	0.3- to 0.9-m/year erosion	0.3- to 0.9-m/year accretion (mostly updrift fillet)	0.3- to 0.6-m/year erosion

Source: Scaled from Figure 4-3 in Crowell and Leatherman (1985)--Note that the accuracy of maps made in the 1830s is questionable due to the lack of cartographic standards (Shalowitz 1964). Therefore, shoreline change statistics based on the 1834/38 charts must be used with caution.

All tide gauges near Long Island have recorded a rise in rsl during this century. As examples, tide level curves for New York City and Montauk are plotted in Figure 3, and Table 3 lists rsl trends at four stations near Long Island. The New York station, located at the Battery at the southern tip of Manhattan Island, is a remarkably long, 122-year record showing an average 2.72 mm/year (0.0089 ft/year) rise in rsl. This means that over the 63 years since the 1933 U.S. Coast and Geodetic Survey (USC&GS) hydrographic data were collected off Shinnecock Inlet, a span less than the lifetime of some of Long Island's inhabitants, the sea has risen about 0.17 m. Even without including sediment losses due to overwash, assuming a beach slope of 1:20, a 0.17-m rise in water level translates to a 3.4-m horizontal movement. This is slightly greater than the retreat rate calculated by Crowell and Leatherman (1985) for the east end of Westhampton. At all four stations listed in Table 3, the 1950 to 1993 trend suggests that the rate of sea level rise has decreased slightly compared to the longer-term average, but at this time we cannot speculate whether such a decreasing trend will continue.

In summary, geologic studies and historic evidence from maps and photographs verify that the Long Island barriers have retreated during the Holocene era. It seems likely that the barrier retreat has been largely a result of rising rsl. Sea level is still rising in this area, and all

indications point to continuing barrier retreat along southern Long Island except in isolated spots of convergence, such as updrift of inlet jetties or at the west tip of Fire Island.

Table 3
Relative Sea Level Trends Near Long Island

		Entire Record Series				1950 - 1993		
NOAA Station	Name	Years of record used	Trend mm/year	Error mm/year	Variability mm/year	Trend mm/year	Error mm/year	Variability mm/year
8510560	Montauk, NY	39	1.85	0.35	28.09	1.78	0.38	28.56
8518750	New York (the Battery)	122	2.72	0.07	28.62	2.27	0.34	28.40
8516990	Willets Pt, NY (LI Sound)	62	2.33	0.22	31.54	1.78	0.40	33.35
8531680	Sandy Hook, NJ	61	3.84	0.22	30.04	3.15	0.37	31.12

Source: Statistics computed by National Oceanic and Atmospheric Administration (NOAA), downloaded from NOAA Internet home page 8/6/1996.

Sediment Grain Size Characteristics

The most comprehensive sediment sampling program along the south coast of Long Island was conducted in the 1950's by the Beach Erosion Board (Taney 1961a). Overall grain sizes along the south shore of Long Island decrease in size from east to west. Taney (1961b) reports that the coarsest material is found in the headland zone extending about 7 km west of Montauk Point. From there west to Shinnecock Inlet, the average median diameter lies between 0.4 and 0.5 mm. Between Shinnecock and Moriches, the sizes display considerable variation (possibly due to gravel bands or patches). Near Shinnecock, sands are slightly coarser than further downdrift to the west. Statistics based on many samples collected along the barrier portion of the south shore (Coney Island to east end of Shinnecock Bay) are listed in Table 4.

Some limited grain statistics were provided by Great Lakes Dredge & Dock, Oakbrook, IL. For the beach west of Shinnecock inlet (exact location unspecified), they reported fine sand with a trace of gravel, d_{50} = 0.74 mm and d_{85} = 3.00 mm. This is surprisingly coarse, and it is likely that the sample statistics were heavily influenced by the gravel portion. For underwater samples below -3.3 m, fine to medium sand had d_{50} = 0.43 mm and d_{85} = 0.80 mm, while above the -3.3-m depth, d_{50} = 0.30 mm and d_{85} = 0.74 mm. More samples were collected in 1995. Grain size distribution curves for each sample at the profile immediately west of the inlet are shown in Figure 4.

Table 4
Sediment Grain Sizes Compiled from Samples Collected Along Barrier Section of South Shore of Long Island[1]

Zone	Average d_{50} (mm)	Maximum d_{50} (mm)	Minimum d_{50} (mm)
Backshore			
Dune	0.32	0.40	0.25
Berm	0.39	0.56	0.25
Average	0.36	0.48	0.25
Foreshore			
mhw	0.38	0.57	0.26
Mid-tide	0.39	1.30	0.18
mlw	0.57	1.70	0.30
Average	0.45	1.19	0.25
Offshore			
-0.6 to -1.8 m	0.30	0.84	0.15
-2.0 to -2.7 m	no samples		
-3.0 to -3.7 m	0.33	0.56	0.16
-5.5 to -7.3 m	0.18	0.61	0.12
-9.1 to -11.9 m	0.19	0.85	0.12
-12.1 to -50 ft	0.29	0.79	0.08
Average	0.26	0.73	0.13
South shore barrier grand average	0.36	1.27	1.06
Shinnecock Inlet[2]			
Throat area	0.43	0.77	0.27
Ebb delta	0.37	0.90	0.21

[1]Condensed from Tables 2 and 5 (Taney 1961a,b).
[2]After McCormick (1971).

McCormick (1971) conducted a detailed sampling program in the inlet and on the ebb shoal for the Town of Southampton. Sediment ranged in size from 0.2 to 0.9 mm. The coarser sizes were restricted to the axis of the inlet and in a zone extending westward from the mouth. The sand became progressively finer offshore. Coarse sand and gravel, found in the center of the inlet, appeared to be encrusted by marine growth, suggesting that it was not often mobilized. Histograms showed that the most common grain size on the ebb shoal was in the band from 1.3 to 1.5 phi (0.41 to 0.45 mm). While the mean grain size of the flood and ebb shoals was nearly the same as on the adjacent beaches, the deltas tended to have a broader distribution of sizes.

McCormick (1971) concluded that sand in the ebb and flood shoals had the correct textural and size properties to be used for beach nourishment.

On most beaches, sediment size varies, with dunes being fine and the swash zone coarse. Sands generally get finer in the surf zone. Therefore, reporting an “average” d_{50} may mask a wide range of grain sizes. At Shinnecock, a value of $d_{50} = 0.40$ mm should be suitable as an “average grain size” to use in engineering calculations.

Shinnecock Inlet History

Charts of Long Island and the approaches to New York Harbor and historical documents note the irregular existence of openings in the barrier between Shinnecock Bay and the Atlantic Ocean. Since there was little urban development in eastern Long Island before the mid-1800's, records are discontinuous, and it is impossible to chart the exact times and locations where inlets have existed. Before the middle of the 20th century, little scientific study had been devoted to the geology and dynamic processes of beaches, and even Fuller's (1914) highly detailed U.S. Geological Survey (USGS) Professional Paper 82, *The Geology of Long Island,* devoted only three pages to beaches and marine deposits. Limited evidence suggests that these old inlets opened as the result of major storms and then closed naturally. Some appear to have remained open for decades, while others closed within months. Details from various geologic studies are chronologically summarized in Appendix A.

USC&GS charts from 1889 and 1890 (Leatherman and Joneja 1980) provide evidence of several inlets into Shinnecock Bay, but all had closed by 1891 (Figure 5). One of the former openings was opposite Shinnecock Neck. Another was slightly west of Ponquogue Point, while two others were east and west of Gull Island, opposite East Quogue. The USGS maps of 1903 and 1904 (Sag Harbor Quandrangle) show no inlets into either Moriches or Shinnecock Bays (Leatherman and Joneja 1980). Fuller (1914) stated that at the time of writing, Shinnecock Bay had no direct connection with the ocean. He also provided an interesting historical note, unfortunately not documented: "An artificial cut made to the ocean was soon closed by the waves." In the late 1930's, the barrier adjacent to Shinnecock Bay was continuous, and a paved road crossed the site of the present inlet. A shoal area about 1,000 m wide paralleled the exposed beach except for a narrow channel which connected deep water in the bay with an indentation in the barrier beach (Nersesian and Bocamazo 1992). Possibly the location of a former inlet, the island breached at this spot during the 1938 hurricane.

The present Shinnecock Inlet was formed as a result of waves and extremely high water during the Great New England Hurricane of 21 September 1938. This hurricane, one of the most destructive storms to strike New England, killed 600 people and devastated coastal communities in Long Island, Connecticut, Rhode Island, and Massachusetts (Allen 1976; Minsinger 1988) (Figure 6). Four openings formed as a result of the storm: one near Warner's Islands, 0.8 km east of Ponquogue Point; a second opposite Cormorant Point; a third opposite the Shinnecock

Hills; and a fourth opposite the Shinnecock Indian Reservation. Three of the breaches closed by the end of 1938, but one stabilized and continued to widen until it was over 200 m across in 1939. In 1941, the inlet was 300 m wide, an inner and outer bar had formed, and a tortuous channel connected the Atlantic with Shinnecock Bay. Although in places the channel was over 6 m deep, the controlling depth was only about 1.2 m.

Construction and Project Work

Various revetments and jetties have been built at Shinnecock Inlet since 1939. To stabilize the shore and reduce inlet migration, the first bulkhead was built by Suffolk County along the west side of the inlet in 1939. The bulkhead deteriorated and a 243-m stone revetment and 40-m groin were built in 1947 by local and State agencies. Stone rubble-mound jetties were finally built during 1953 and 1954 by the State of New York, Suffolk County, and the Town of Southampton. The east jetty was 415 m long and the west 260 m. The jetties deteriorated over time, with much stone loss occurring at the tip of the east jetty. Though the inlet became a Federal project in 1960, the jetties were not rehabilitated until 1992 and 1993. Chronological details of construction are listed in Appendix A.

Flood Shoal Development

The flood shoal experienced slow growth from its beginning in 1938 until 1953, when construction of the jetties began (McCormick 1971). Between 1950 and 1955, the shoal grew rapidly, approximately doubling its size. The rapid growth was caused by the increasing size of the inlet, but growth slowed after 1955 because of the gradual constriction of the size of the tidal channels that crossed the flood delta. The west portion of the shoal was stabilized by the spread of salt-marsh grasses. While the west area was stable after the mid-1950's, the northern margin of the shoal continued to grow into the bay. McCormick estimated the flood shoal growth rate between 1955 and 1969 to be 45,000 m^3/year (59,000 yd^3/year). We have no data to verify if the shoal has continued to grow.

Ebb Shoal Development and Inlet Morphology

1933 Morphology

The most complete pre-inlet regional hydrography was collected by the USC&GS in 1933 (Atlantic Ocean: charts H-5324 and H-5325; Shinnecock Bay: chart H-5323 (Leatherman and Joneja 1980)). These data are conveniently available from the National Geophysical Data Center on CD-ROM and have been used in this report to depict the baseline conditions in the area. The 1933 tracklines are not as tightly spaced as lines in more modern surveys but still are surprisingly

comprehensive considering that, at least in shallow water, measurements were made with sounding poles or lead lines (Shalowitz 1964). Figure 7 shows the 1933 survey lines, and Figure 8 is the same data contoured. In these figures, a modern shoreline has been included for reference, but the reader must remember that Shinnecock Inlet was not open then. This shoreline, also shown in subsequent figures, is based on 1990's National Oceanic and Atmospheric Administraction (NOAA) charts. Notes on these and other data sources are listed in Appendix B.

In 1933, the Atlantic shoreline was almost straight and showed no obvious evidence of older inlets. From the shore to about 7 m, a series of bars is evident in the contoured bathymetry. Deeper than 7 m, offshore contours are reasonably straight and parallel.

1949 Morphology

Shinnecock Inlet and its ebb and flood shoals were surveyed by USACE in July and August of 1949. By this time, State of New York, Suffolk County, and the Town of Southampton had built a 240-m stone revetment on the west side by the inlet. Only 11 years after the inlet was breached, a broad, oval-shaped ebb shoal had already formed (Figure 9). It extended about 1,500 m to the west, 400 m offshore, and at least 600 m to the east of the inlet's mouth (the survey did not extend far enough east to cover the full shoal). The top of the shoal was at a depth of about -3 m, and the bar front dropped steeply from -3 m to the seafloor beyond -6 m. In the flood shoal, two dredged channels are evident, one extending from the landward end of the inlet to the west and another extending northeast and then north.

1994 Morphology

In June and August of 1994, Shinnecock Inlet and the ocean coast between Moriches and Shinnecock Inlets was surveyed with the SHOALS[1] helicopter-borne hydrographic LIDAR survey system. The tremendous data density recorded by the SHOALS system provided unprecedented seafloor detail. Unfortunately, the 1994 surveys were not flown far enough seaward to cover the entire ebb shoal. The contoured data (Figure 10) show that the ebb shoal attached to the downdrift shore about 2.4 km west of the west jetty. The shoal platform had depths about 3 m below mllw. The deep area seaward of the jetties was the deposition basin from which 363,000 m^3 (475,000 yd^3) of sand was removed in early 1993. Because coverage did not include the edge of the ebb shoal, these 1994 data were not used for volumetric comparisons.

[1]Scanning Hydrographic Operational Airborne LIDAR Survey (SHOALS) is a USACE- developed system using a helicopter-based laser to conduct hydrographic surveys (Lillycrop, Parson, and Irish 1996).

One unusual morphologic feature evident in the 1994 data is a deep pit - almost a channel - about 300 m west of the west jetty trending approximately in a north-south direction. The deep part of the pit, -6.1 m mllw, extended to within 150 m of the shore. The pit is immediately offshore of the portion of the beach experiencing the greatest erosion.

1996 Morphology

The SHOALS helicopter system was mobilized to survey Shinnecock Inlet again in May and June 1996. This survey covered the ebb shoal more completely than the 1994 survey, as shown in Figures 11 and 12.

The shoal was shaped in the form of two irregular lobes that flanked the mouth of the inlet. The east lobe was narrow and projected seaward parallel to the east jetty. The west lobe was approximately triangular-shaped, with the seaward edge dropping off to the south. The bar front on this west lobe was marked by closely spaced contours that extended from about -3.0 to -6.7 m mllw. The west end of the lobe approached the shore about 1,800 m west of the west jetty.

A north-south channel ran from the mouth of the inlet seaward between the two lobes. This is the area that was dredged as a deposition basin in 1993, which in 1996 was still deeper than the surrounding ebb shoal lobes. Seaward (south) of the channel, the shoal dropped off into deep water, with the bar front extending from -9.1 to -12.2 m mllw. The distance from the end of the jetties to the seaward edge of the shoal was about 1,100 m.

The deep pit adjacent to the west jetty, described in 1994, was still present. Depths greater than -5.5 m mllw are found within 150 m of shore. Some physical process appears to be causing erosion and sediment transport in this area. Possibly, waves are refracted at the edge of the shoal and concentrate their energy in this area. Other possibilities include sand depositions in the reach immediately downdrift from the jetties and erosion by a tidal eddy in that area. The pit extends perpendicular to the shore and does not resemble marginal flood channels found at many other inlets (these are typically parallel to the shore and channel the flood tide into the inlet mouth). Without current or drifter data in this area, these hypotheses cannot be tested. Nevertheless, it is likely that the linear pit and the processes which are maintaining this deep area are related to the serious erosion experienced along the adjacent shore.

Erosion west of the inlet

The shore west of the west jetty has been eroding since the jetties were completed in 1953. Between 1954 and 1974, while the updrift fillet was filling with sand, the beach fronting Dune Road receded from about 213 m seaward of the road to 137 m (USAE District, New York 1988). Once the fillet east of the east jetty was largely filled around 1974, additional littoral material bypassing the inlet temporarily stabilized the zone west of the inlet. But by 1978, erosion began to increase again, and by March 1983, there remained only 91 m of beach fronting Dune Road.

In the Shinnecock General Design Memorandum (USAE District, New York 1988), two reasons are cited for the continuing erosion: First, the growth of the sandbar across the inlet absorbed about 76,500 m^3/year (100,000 yd^3/year) of the littoral material. Second, the growth of the bar caused the ebb flow to be diverted more to the southwest and closer to the beach.[1] The loss of sand was estimated to be 46,600 m^3/year (61,000 yd^3/year) between 1955 and 1984 for the 1,800-m stretch of shore west of the inlet.

Volumetric analysis, 1933-1949

The morphology of the 1949 shoal is revealed in a plot based on a volumetric comparison with the 1933 USC&GS (pre-inlet) data. Unfortunately, the 1933 survey only covered the ocean and bay sides of the barrier but not the topography of the barrier where the inlet formed in 1938. The 1926 USGS topographic map displayed dune and marsh features but no elevation data. Therefore, the pre-inlet barrier was simulated by using contemporary topography and assuming that the pre-1938 barrier was similar. Adjustments to convert the 1933 data to a modern datum are described in Appendix C, and the procedure for generating the topography is outlined in Appendix D.

In Figure 13, accretion is shown by green contours and erosion by red. For the ebb and flood shoals, volumes were only computed for areas where the 1933 and 1949 data sets overlapped. Volume changes, computed using Terramodel© software, are summarized in Table 5.

Table 5
Volumetric Comparison, 1938 - 1949[1]

Region	Accumulation, m^3(yd^3)	Loss, m^3(yd^3)
Ebb shoal	1,284,000 (1,680,000)	145,000 (190,000)
Barrier and inlet	23,000 (30,000)	960,000 (1,260,000)
Flood shoal	2,220,000 (2,900,000)	495,000 (647,000)

[1]Note: Preinlet data collected in 1933, but inlet opened in 1938.

The total accumulation of sand on the Atlantic side of the barrier was 1,300,000 m^3 (1,700,000 yd^3), while the flood shoal grew 2,200,000 m^3 (2,880,000 yd^3). Since there are no important inland sand sources, the growth of the flood shoal must have been fed by a combination of sand funneled through the inlet from the interrupted littoral drift and from erosion of the barrier after the 1938 breach.

[1]The General Design Memorandum (USAE District, New York 1988) states that these explanations are based on bathymetric surveys. The value of 76,500 m^3/year (100,000 yd^3/year) was computed from 1955, 1984, and 1985 surveys.

The following steps outline the method used to estimate the annual growth rate of the two shoals. We assumed that most of the sand eroded from the barrier moved north onto the flood shoal.

Flood shoal - Barrier erosion = Sand supplied by littoral drift to flood shoal
2,220,000 - 960,000 = 1,260,000 m^3
(2,900,000 - 1,260,000 = 1,640,000 yd^3)

Flood shoal + Ebb shoal = Combined shoal volume supplied by littoral drift
1,260,000 + 1,284,000 = 2,540,000 m^3
(1,640,000 + 1,680,000 = 3,320,000 yd^3)

Combined shoal volume ÷ Years since inlet formed = Rate of **flood & ebb** shoal growth
2,540,000 ÷ 11 = 231,000 m^3/year
(3,320,000 ÷ 11 = 302,000 yd^3/year)

Ebb shoal volume ÷ Years since inlet formed = Rate of **ebb** shoal growth
1,284,000 ÷ 11 = 117,000 m^3/year
(1,680,000 ÷ 11 = 153,000 yd^3/year)

If we assumed that the inlet, at least during the first decade of its existence, formed an almost complete barrier to littoral drift, the growth of the shoals indicate that the **gross** annual longshore transport was in the range of 230,000 m^3/year (301,000 yd^3/year). The growth of the ebb shoal alone understates the total amount of sediment in motion.

Volumetric analysis, 1933 - 1996

a. Data analysis. To evaluate morphologic changes in the region near Shinnecock Inlet, the 1933 USC&GS bathymetry was subtracted from the 1996 SHOALS bathymetry using the VOLUME function in Terramodel© software. The 1933 data were referenced to mean low water (mlw). However, in 63 years, rsl in this area has risen about 0.171 m, based on the annual trend computed by NOAA for the Battery in New York Harbor (Table 3). In other words, the 1933 mlw datum is lower than the contemporary mlw datum, and therefore any individual 1933 depth must be lowered to be directly comparable to contemporary data. We adjusted the 1933 soundings by 0.171 m, a value obtained by multiplying the trend, 2.72 mm/year × 63 years. Note that the adjustment is based on the average trend, but in any one year actual rsl may deviate greatly from the trend due to numerous oceanographic and climatologic factors. Finally, the 1933 depths were lowered another 0.34 m to adjust from mlw to National Geodetic Vertical Datum (NGVD) (1929 adj.) to allow direct comparison with the 1996 SHOALS data. Adjustments are summarized in Equation 1:

$$Z_{(\text{modern mlw})} - Z_{(1933\ \text{mlw})} = 0.171\ \text{m}$$

$$Z_{\text{NGVD}} - Z_{(\text{modern mlw})} = \underline{0.34\ \text{m}} \tag{1}$$

$$Z_{\text{NGVD}} - Z_{(1933\ \text{mlw})} = 0.511\ \text{m}$$

b. Morphologic changes. By contouring the volume changes from 1933 to 1996, we can clearly see how the semicircular ebb tide shoal has grown seaward of the mouth of Shinnecock Inlet. In Figure 14, the green contours represent the sand that has accumulated on what was a relatively smooth seafloor in 1933. The shoal projects seaward at least 1,400 m from the shore. The contours at the seaward edge of the data coverage show a 1.8- to 2.4-m thickness, suggesting that the ebb shoal must extend seaward a considerable distance further, possibly 100 m or more. The thickest sand accumulations, about 8 m, are southwest of the jetties 1,000 m offshore. The shoal welds to the downdrift shore about 1,200 m west of the west jetty. Figure 14 also reveals that considerable scour has occurred between and immediately seaward of the jetties. Sand loss exceeds 8 m in the deepest part of the channel.

c. Volumes. The total accumulation of sand over 58 years (1938 to 1996) within the area bounded by the box in Figure 14 was 6,410,000 m^3 (8,381,000 yd^3). This represents average ebb shoal accretion of 110,000 m^3/year (145,000 yd^3/year). This value understates the total sediment transport in the area because not all longshore transport is trapped on the ebb shoal; some proportion is certain to be bypassing the shoal and continuing on down the coast. In addition, in 1993 the NAN removed 363,000 m^3 (475,000 yd^3) from the deposition basin, a significant man-made loss from the local shoal and inlet system. The computed annual accretion of 110,000 m^3 (145,000 yd^3) is similar to recent transport estimates from NAN and MME, which are discussed later.

Summary

The ebb shoal growth rates for the two time periods (1933 to 1949 and 1933 to 1996) indicate a similar and possibly relatively constant annual accumulation (117,000 m^3/year versus 110,000 m^3/year, respectively). This is supported by the linear nature of the plot shown in Figure 15, which has a correlation coefficient of 0.99. Even though the ebb shoal may be approaching equilibrium (see later section), no change in the rate of growth is apparent (Table 6).

Table 6
Summary–Ebb Shoal Growth

Period	Total Accumulated Volume (m^3)	Annual Growth Rate (m^3/yr)
1938-1949[1]	1,284,000	117,000
1949-1996	5,126,000	109,000
1938-1996	6,140,000	110,000

[1]Pre-inlet data collected in 1933, but inlet opened in 1938

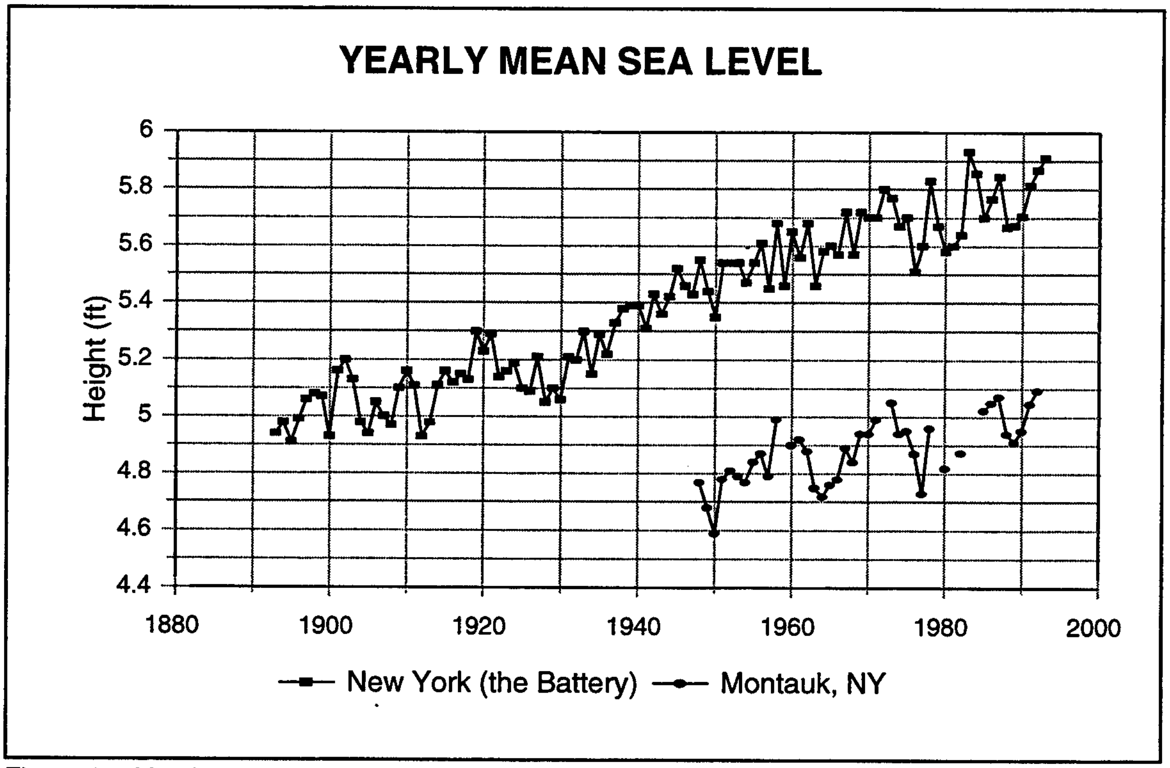

Figure 3. Yearly mean sea level at two stations near Long Island. Data from NOAA Internat home page

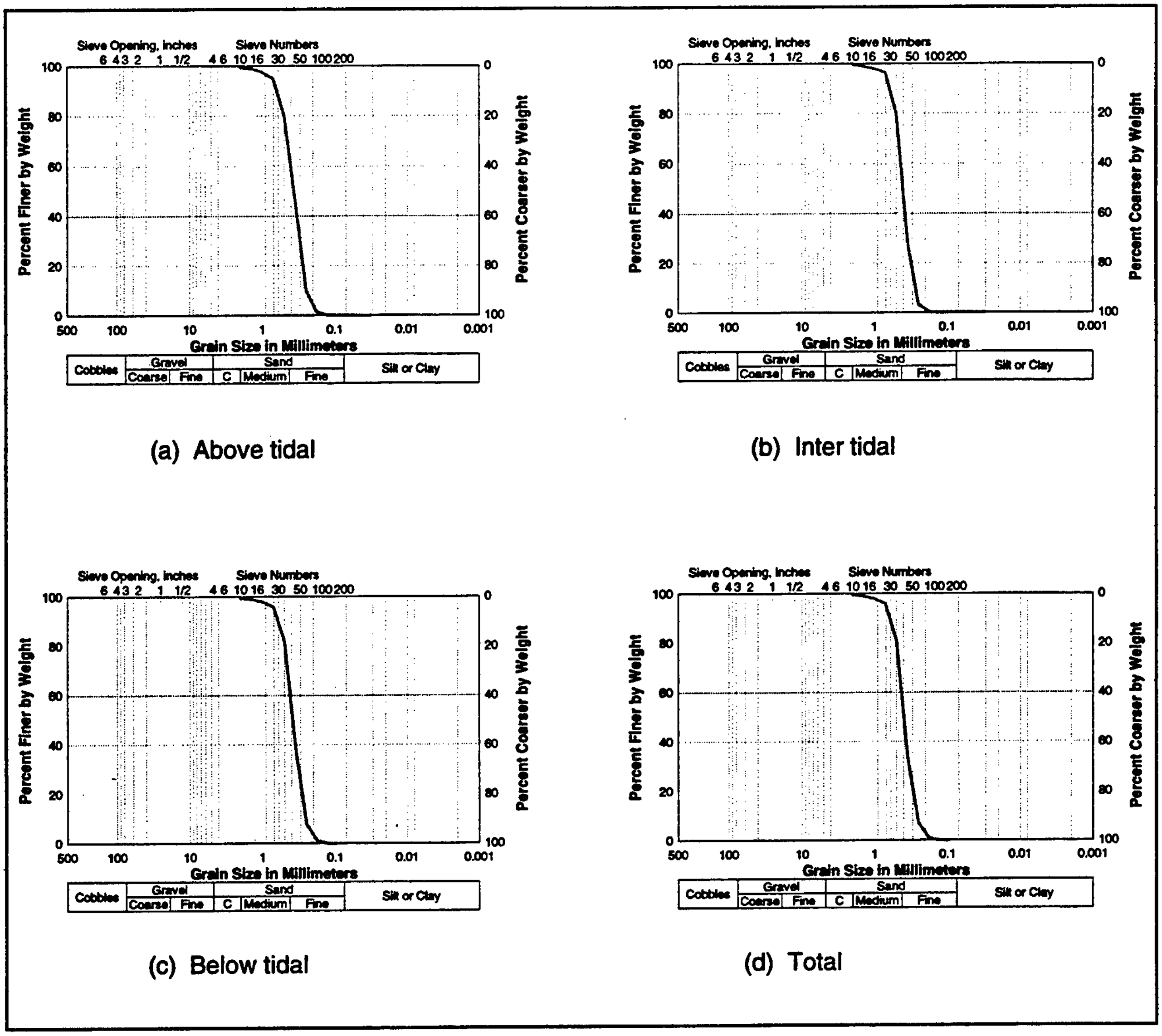

Figure 4. Grain size distribution curves for beach west of inlet

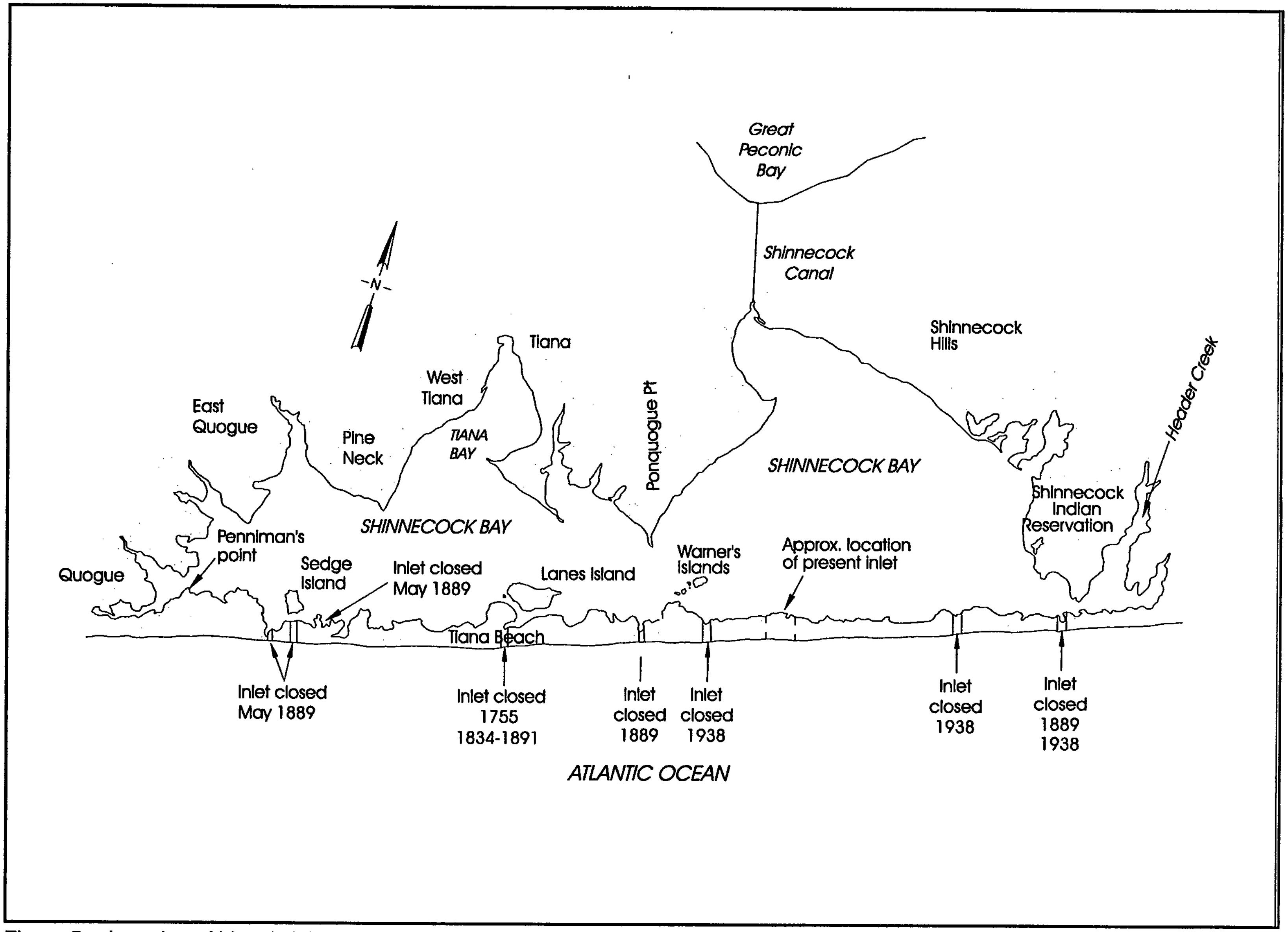

Figure 5. Location of historic inlets opening into Shinnecock Bay. Based on 1889-1890 USC&GS charts (modified from Leatherman and Joneja 1980). Water area of Shinnecock Bay is 38.0 km^2

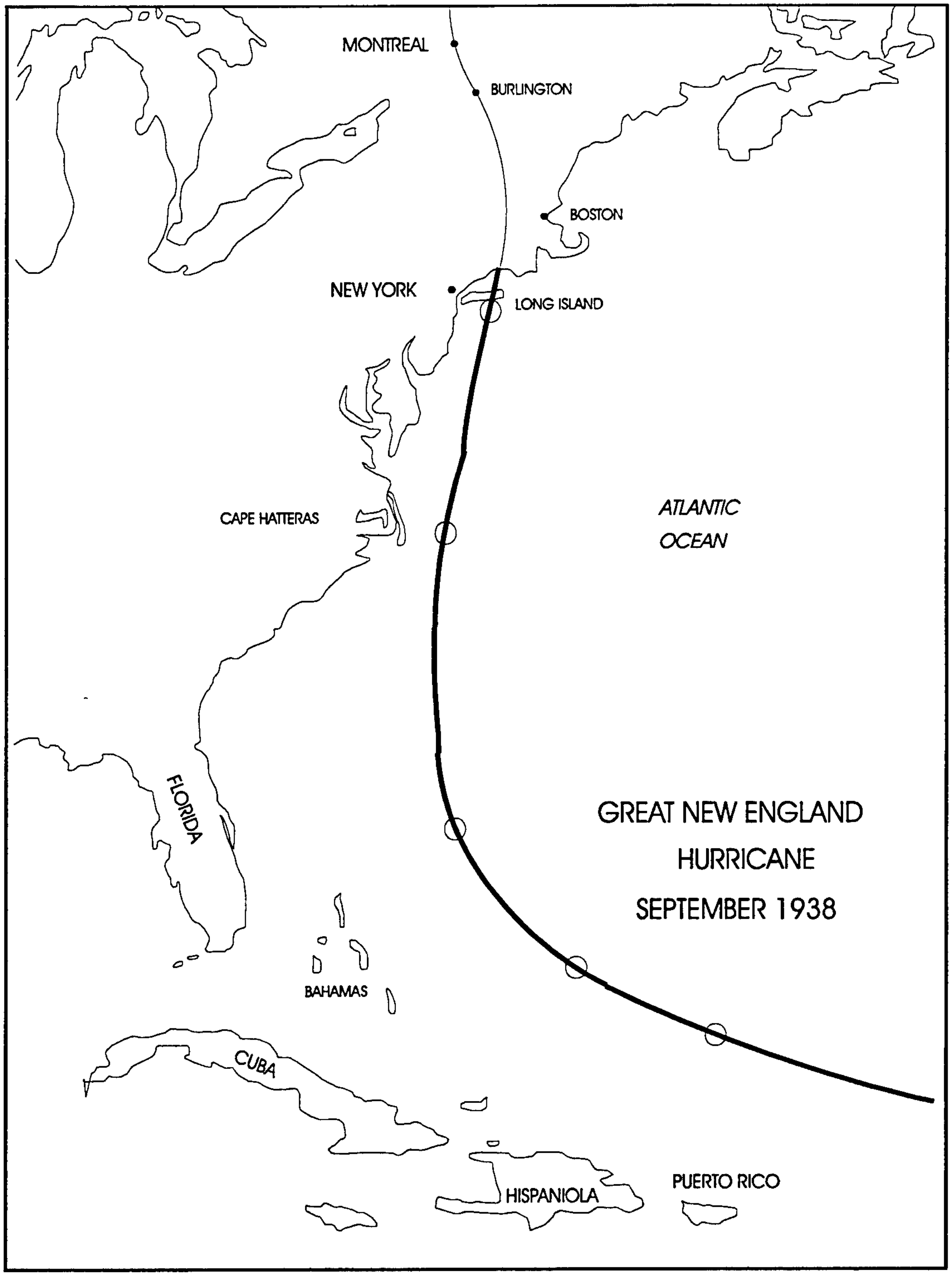

Figure 6. Storm track of the Great New England Hurricane that made landfall on Long Island on September 21, 1938 (from modified *Providence Journal* (1938))

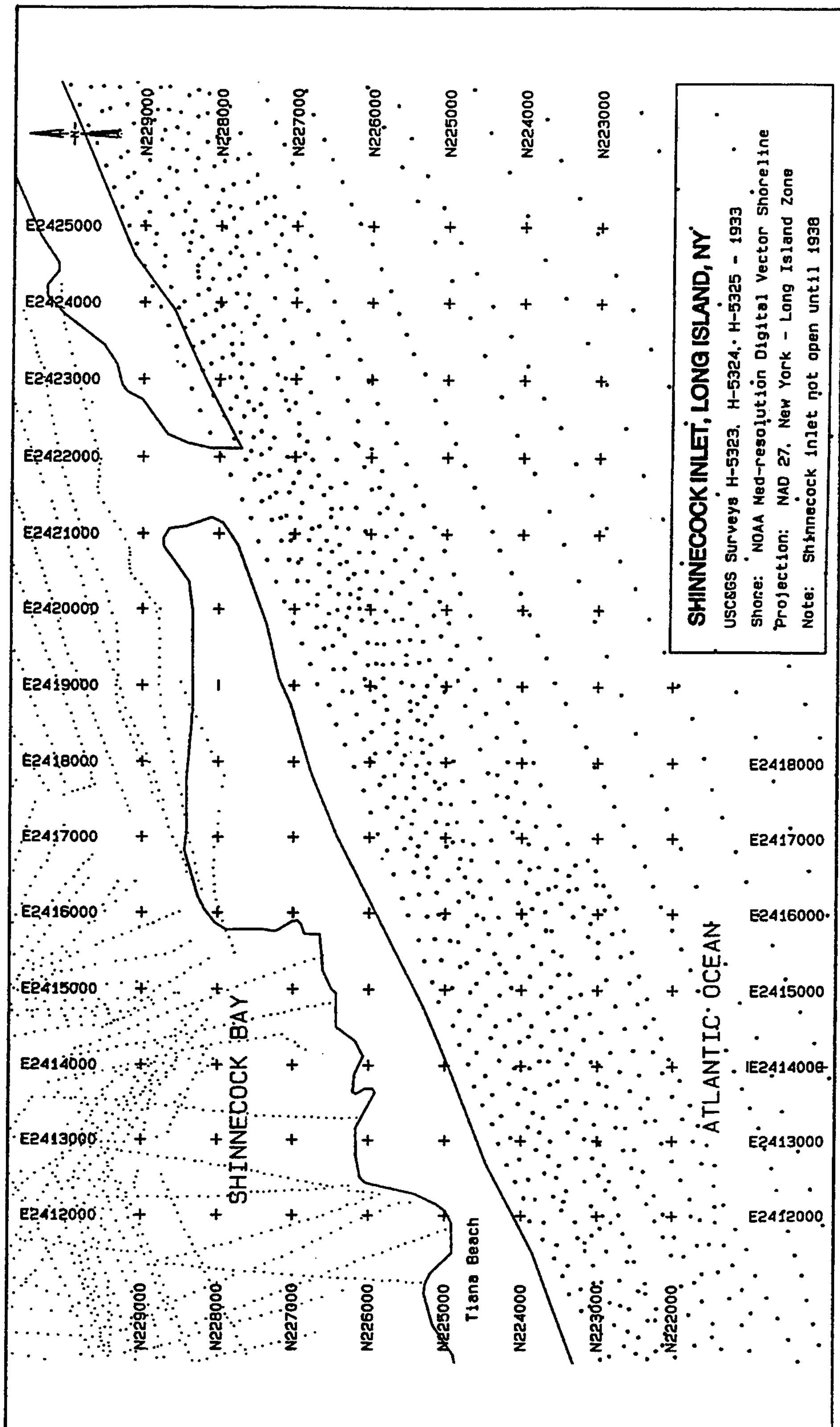

Figure 7. Track lines of USC&GS 1933 charts H-5323, H-5324, and H-5325. Shinnecock Inlet was not open at that time, shoreline is recent, and inlet position is shown for reference

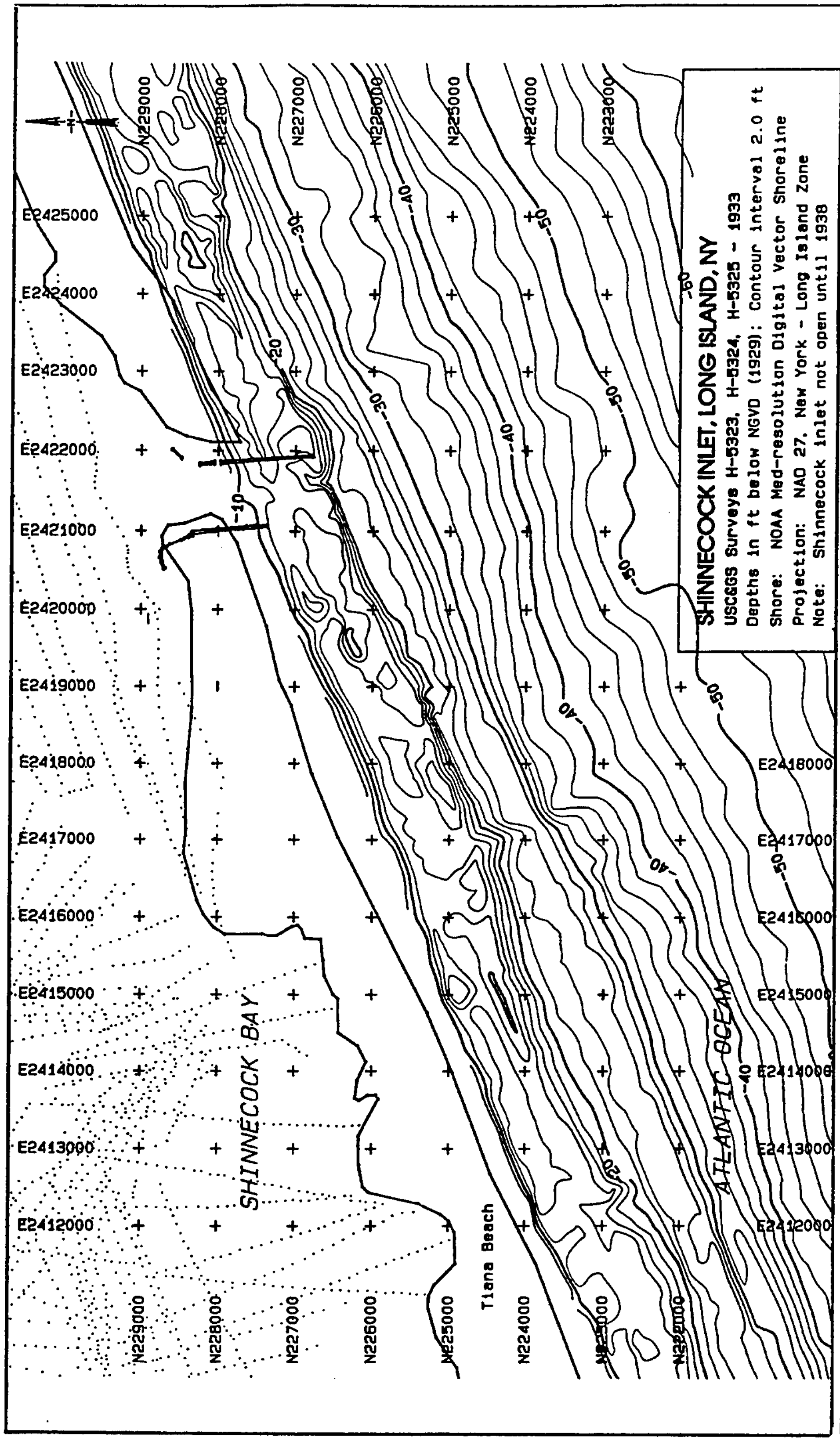

Figure 8. Contoured bathymetry from 1933 USC&GS charts H-5323, H-5324, and H-5325. Shinnecock Inlet was not open at that time, shore is recent, and inlet position is shown for reference

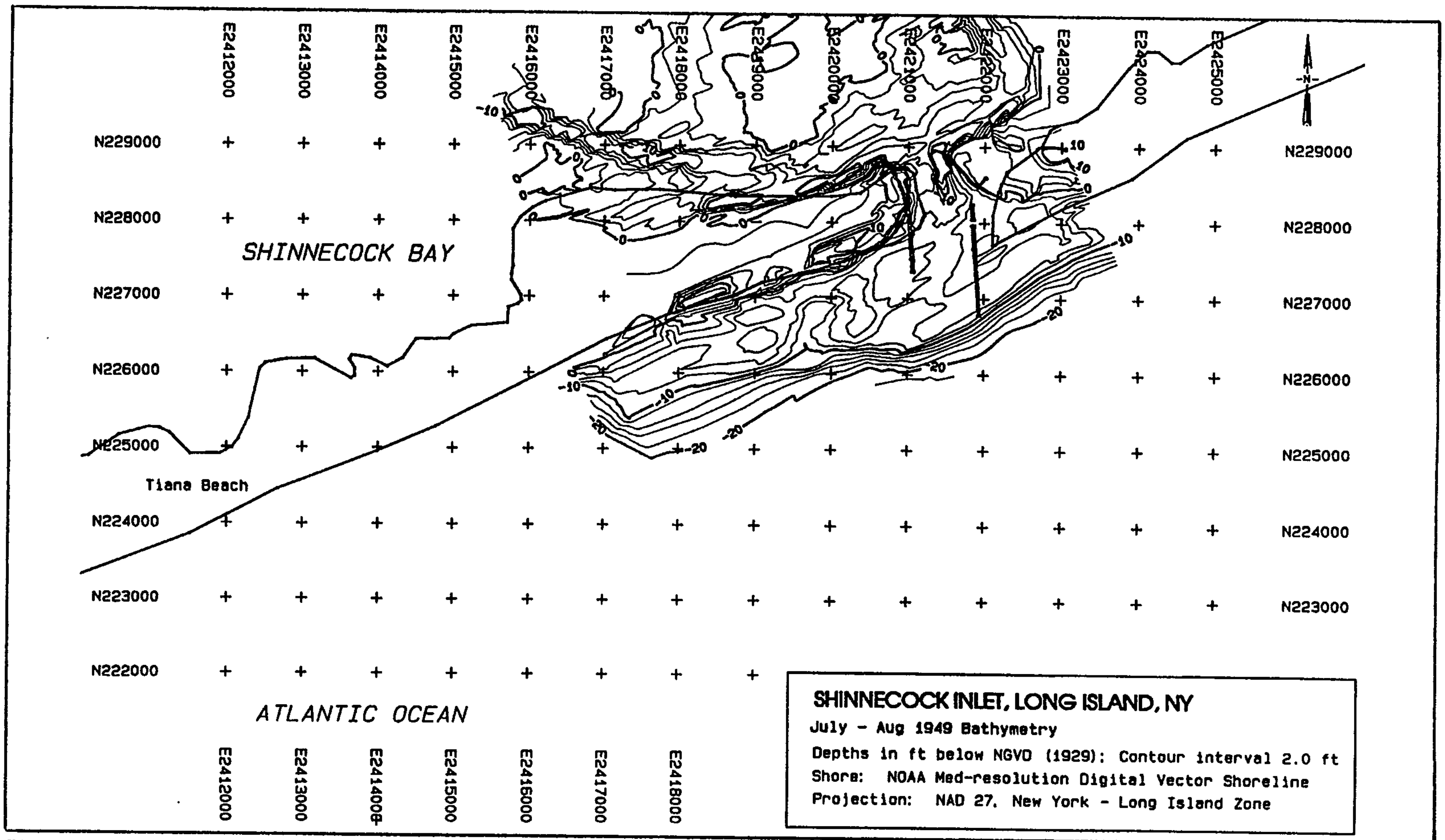

Figure 9. 1949 bathymetric survey. Note that the contemporary jetties are shown for positional reference, although in 1949 they had not yet been built. The ebb shoal already displays wide oval shape that characterizes its shape in the 1980s and 1990s

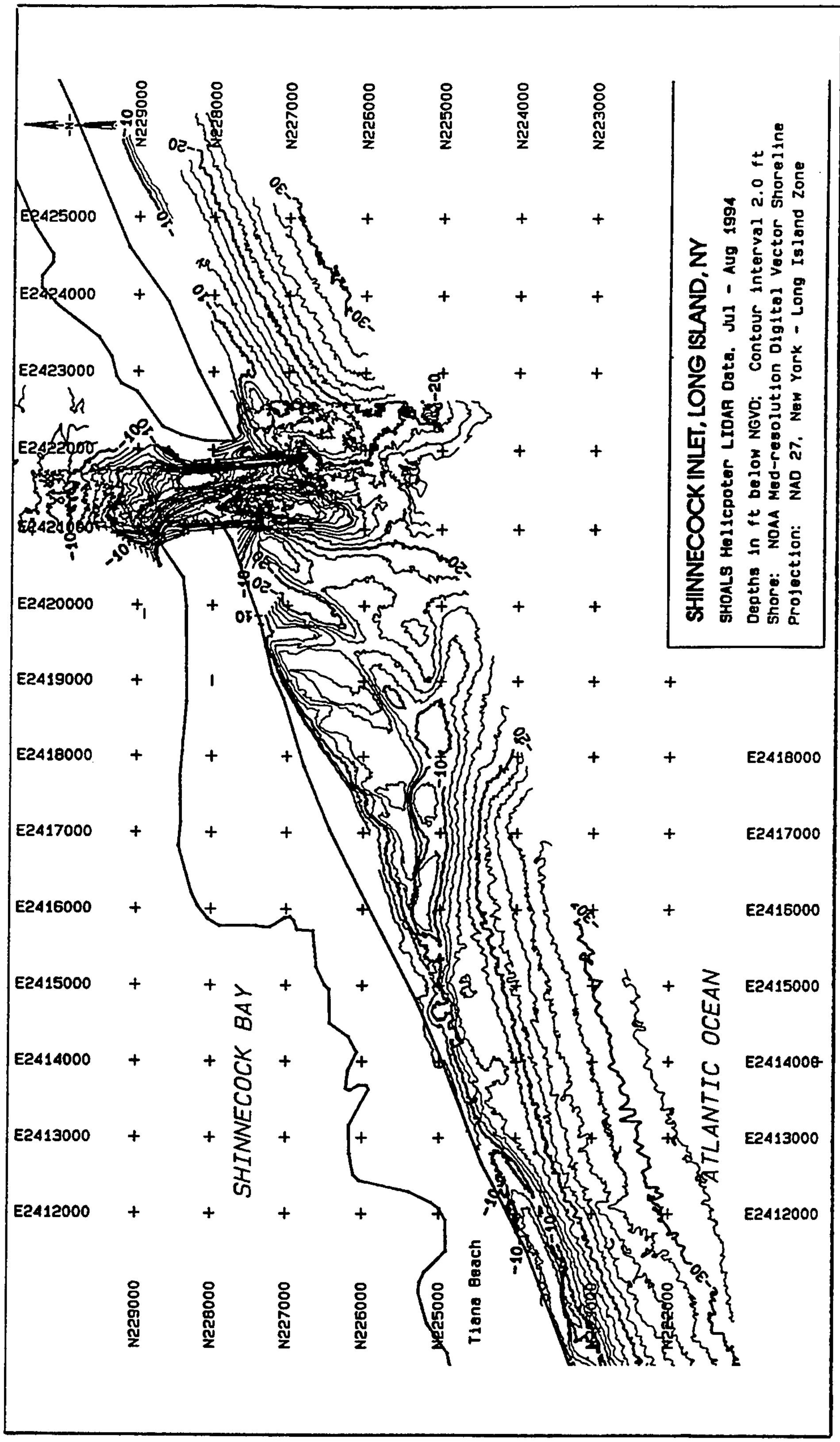

Figure 10. Contoured bathymetry from July and August 1994, SHOALS helicopter LIDAR bathymetry survey. Large scour area with depths greater than 30 ft occurs in the channel near the west jetty

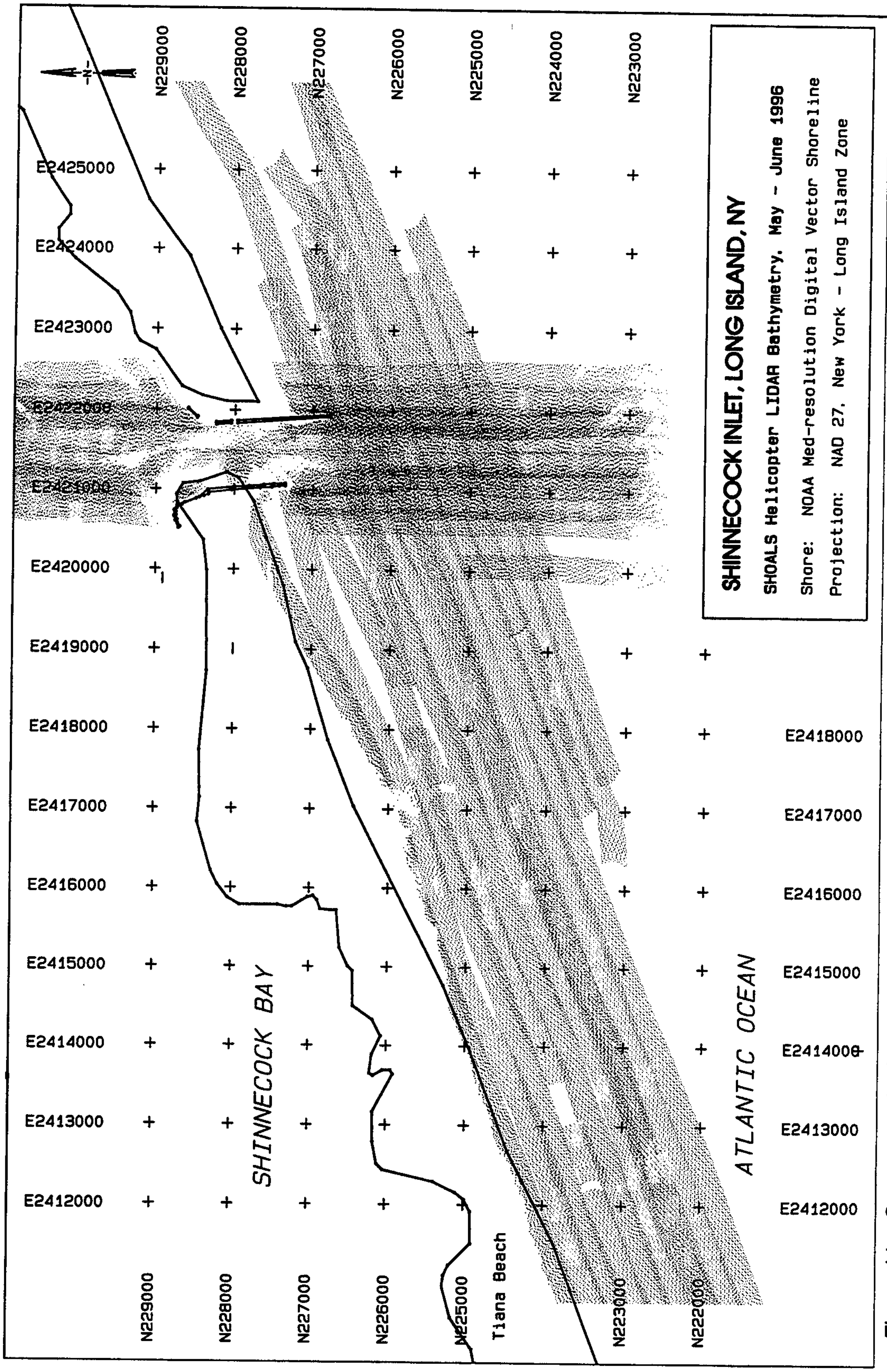

Figure 11. Survey swaths from May and June 1996 SHOALS helicopter LIDAR bathymetry survey. Original coverage in this area included almost 500,000 soundings. To simplify computer manipulation and contouring, the data were decimated to one-quarter of the original number of soundings (this figure shows about 120,000 points)

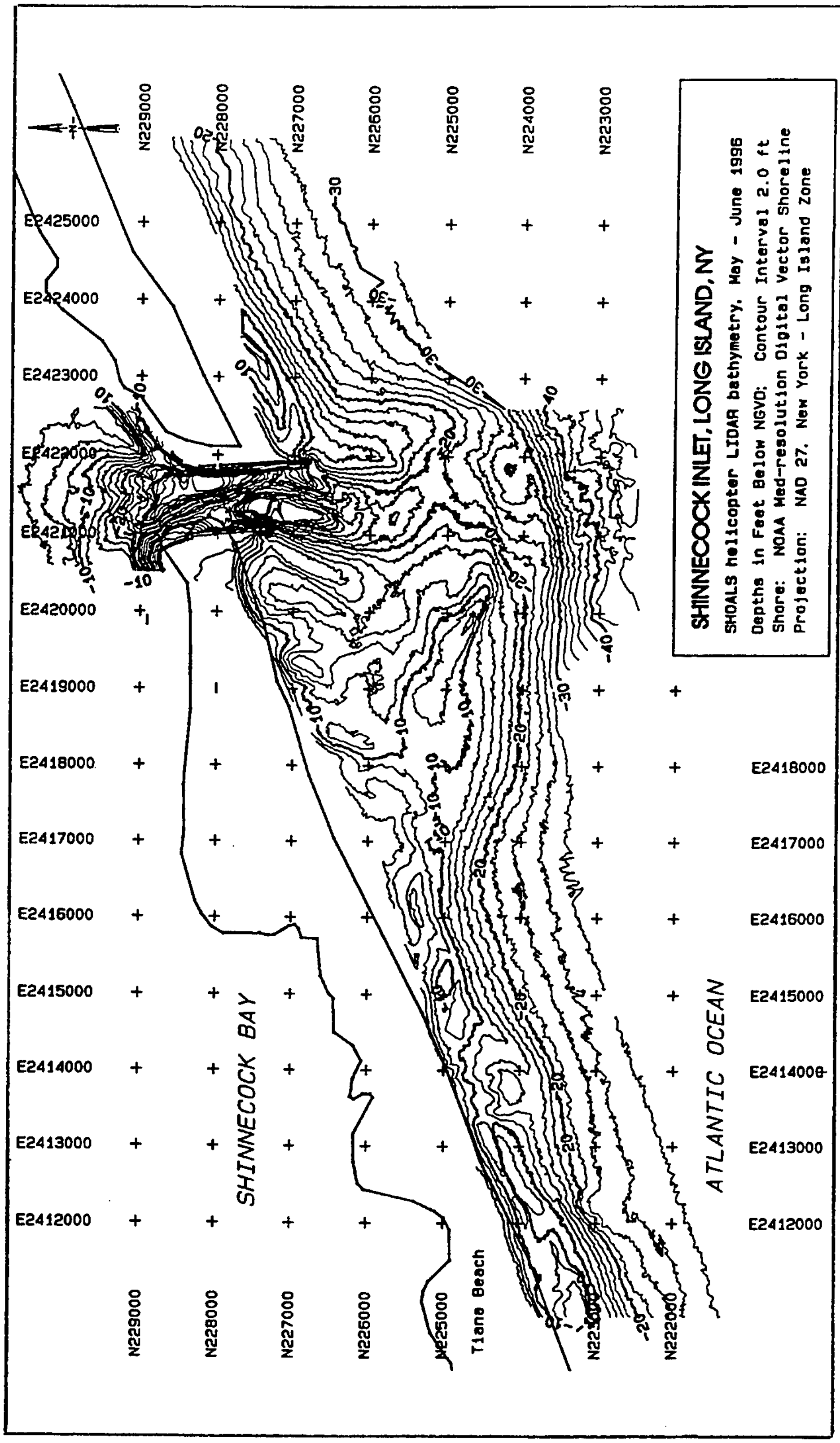

Figure 12. Contoured bathymetry from May and June 1996 SHOALS helicopter LIDAR bathymetry survey

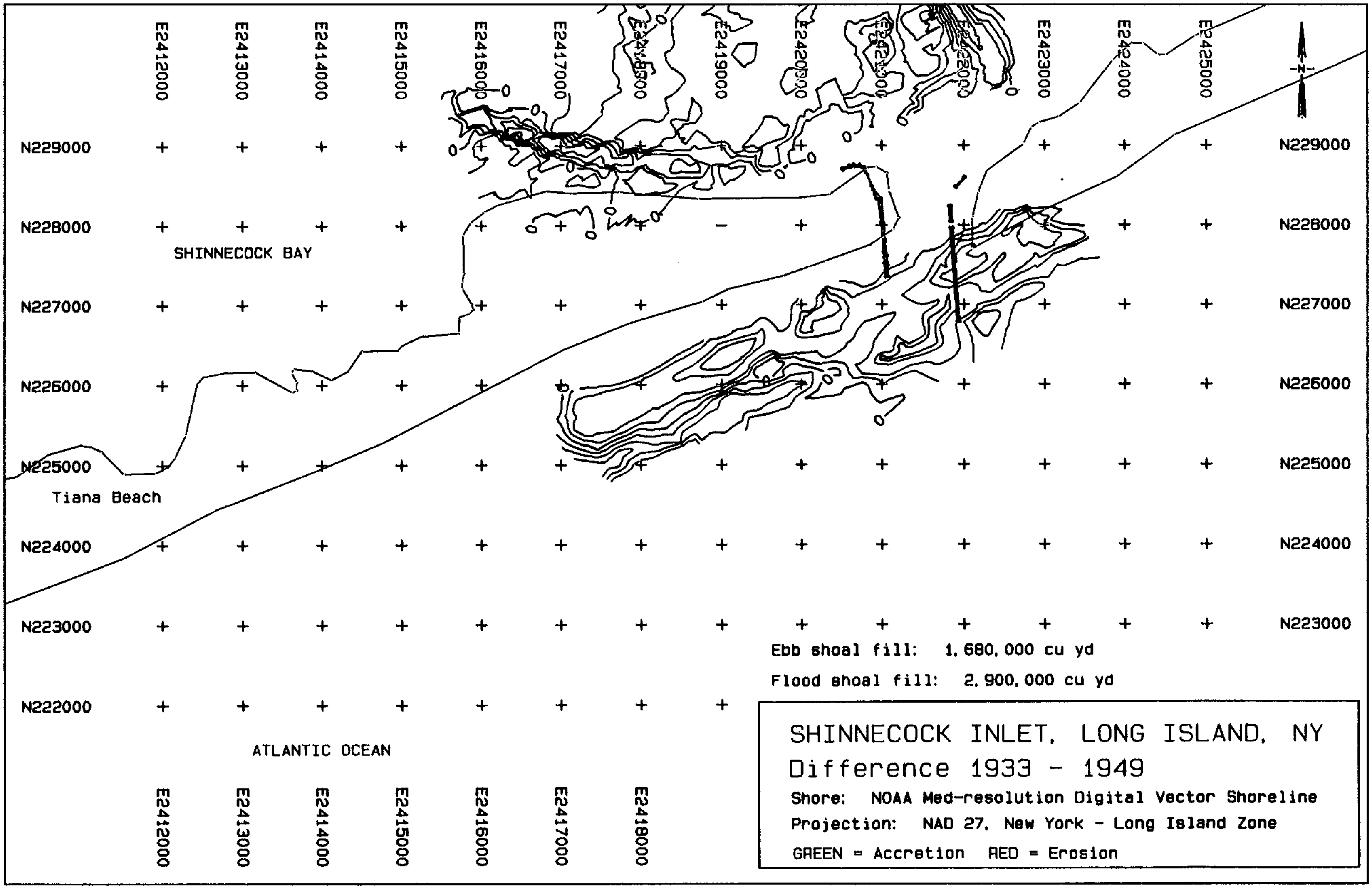

Figure 13. Volumetric changes, 1933-1949. red represents erosion; green is accretion. An elongated ebb shoal formed 11 years after the inlet formed. Contemporary jetties are shown for positional reference, although in 1949 they had not yet been built

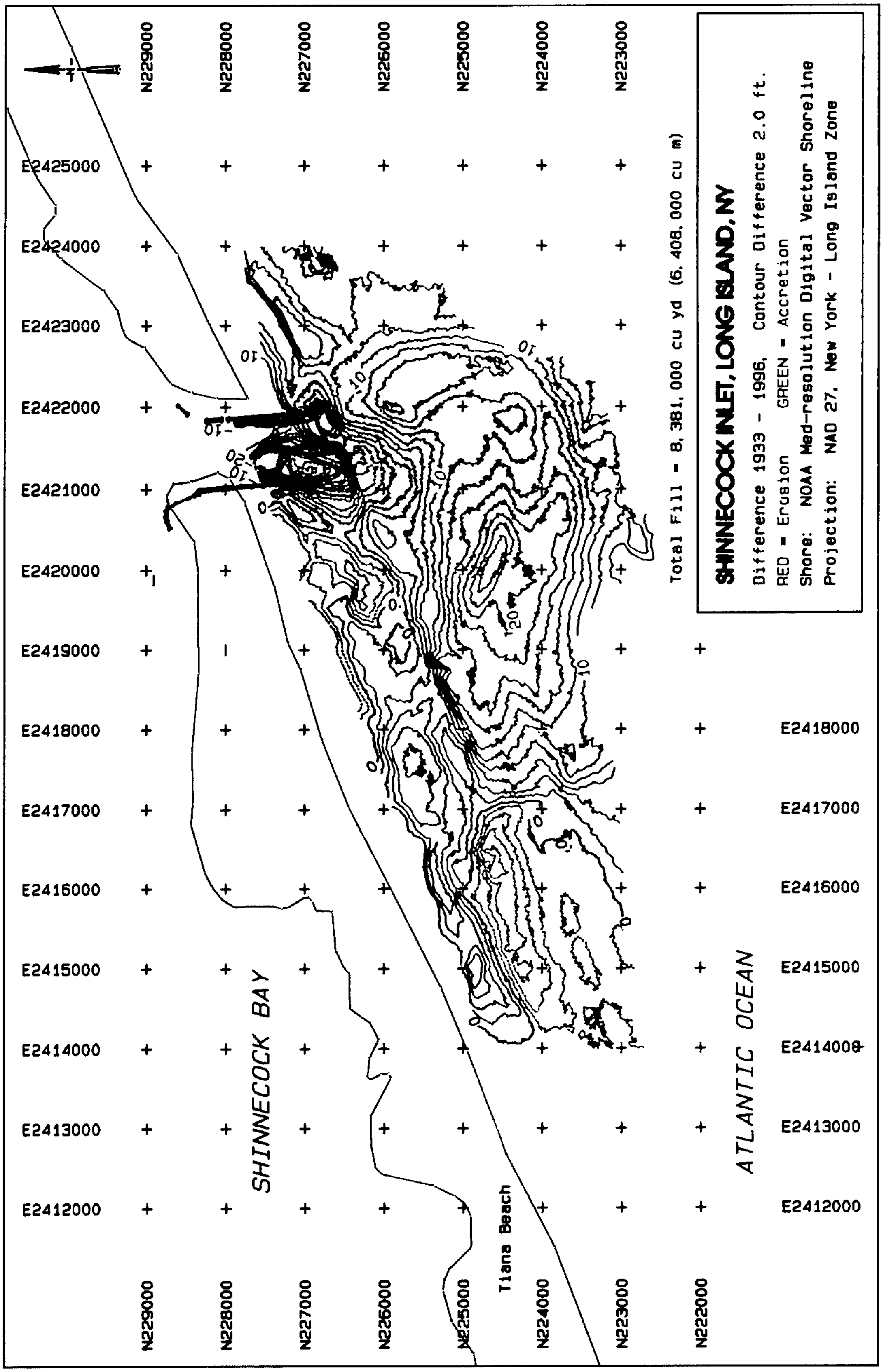

Figure 14. Difference in surface between 1933 USC&GS and 1996 SHOALS surveys. Datum adjustments described in the text

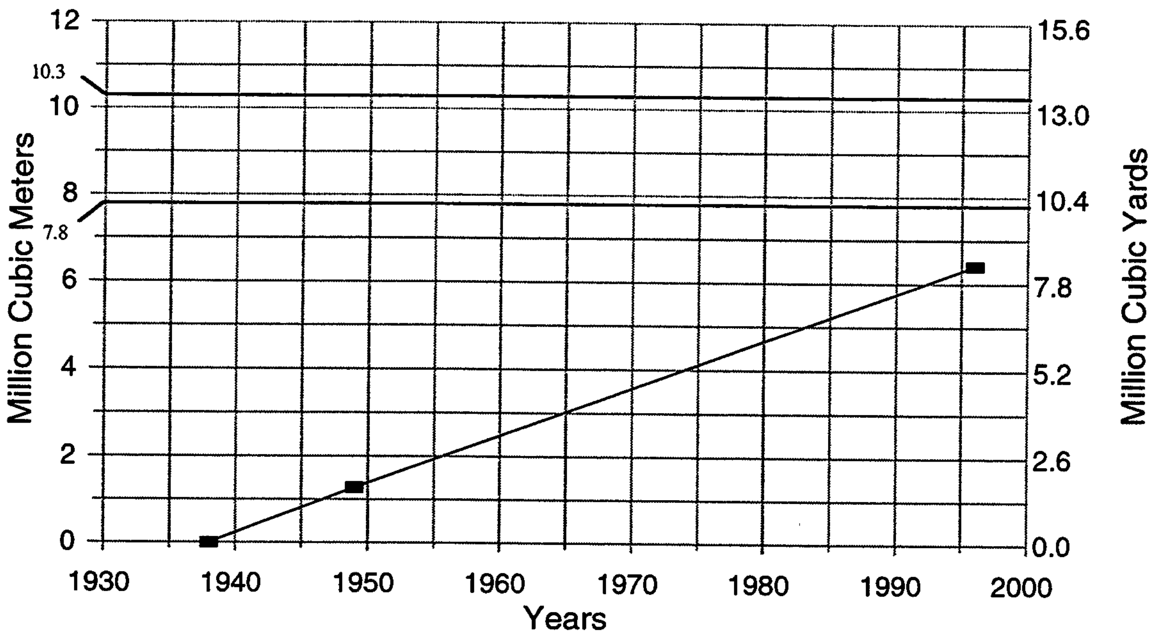

Figure 15. Ebb shoal volume for three dates. Equilibrium ebb shoal volumes are also noted (see Table 13)

3 Physical and Coastal Processes

Climate

The climate of Long Island is characterized by mild winters and relatively cool summers. Extreme fluctuations of temperature are rare due to the moderating effects of the Atlantic Ocean. The mean annual temperature in the project area is approximately 10°C (50°F). The coldest months (January and February) average about -1°C, while the warmest month (July) averages 21°C. Extreme temperatures range from about -23°C to 38°C. The average annual precipitation is approximately 114 cm, with lower amounts in the summer months (Moffatt & Nichol 1996).

Tides

Tides on the south shore of Long Island are semidiurnal with a mean range of 0.88 m at the Inlet entrance and an average spring tide range of 1.1 m (National Ocean Service 1993). Table 7 shows the water level datums for Shinnecock Inlet, relative to mllw equal to 0.00.

Wind setup is a local phenomenon that occurs most dramatically in shallow water. During strong onshore winds, setups of 0.6 to 0.9 m are not uncommon, and extremely high wind velocities coupled with very low barometric pressures, i.e., tropical depressions or hurricanes, have caused tides as high as 2.5 m above mlw (Hurricane Donna, 12 September 1960) in Shinnecock Bay (USAE District, New York 1988).

Currents

In order to investigate scour holes in Shinnecock Inlet, the U.S. Army Engineer Waterways Experiment Station (USAEWES) conducted current studies each year from 1991 to 1994. The 1991 study used an InterOcean S-4 current meter, while the 1992 to 1994 studies used a broad-band acoustic doppler current profiler. The 1991 survey indicated that relatively stronger currents moved along the west side of the inlet during flood tide, while stronger ebb currents

Table 7
Shinnecock Inlet, Atlantic Ocean: Elevations of Tidal Datums Referred to Mean Lower Low Water[1]

Tidal Level	Elevation (m)
Highest observed water (12/25/1978)	2.19
Mean higher high water (mhhw)	1.15
Mean high water (mhw)	1.06
Mean tide level (mtl)	0.56
NGVD 1929	0.38[2]
Mean lower water (mlw)	0.049
Mean lower low water (mllw)	0.00
Lowest observed water level (3/28/1979)	-0.51

[1]Elevations from U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA). Publication date: 7/20/1987 (NOAA 1987).
[2]NGVD based on data from USAE District, New York, surveyors (elevation of 4.13 m (13.54 ft) measured at bench mark No. 1, 1974).

were observed at the east side. The maximum average speed at Shinnecock Inlet was 1.6 m/sec with instantaneous speeds exceeding 2.1 m/sec. The 1991 survey also established two current measuring stations within Shinnecock Bay (S-4 and S-5) to assess the strength and direction of tidal currents at the bay channels north of the inlet (Figure 16). Ebb currents had approximately the same strength at both locations, around 1.3 m/sec. However, flood currents at S-5 were weaker than those at S-4. During flood tides, the currents at S-4 and S-5 both moved in a westerly direction, suggesting that there may be a clockwise circulation in Shinnecock Bay (USAEWES 1991).

The 1992 current study was conducted during an average tidal height condition for the area, while the 1993 survey was planned specifically for a spring tide condition. The peak flood flow in 1992 was about 1,800 m^3/sec (63,600 ft^3/sec), which was 20 percent higher than the peak ebb flow of 1,500 m^3/sec (53,000 ft^3/sec). The 1993 spring tide survey showed a peak flood flow of 2,940 m^3/sec (104,000 ft^3/sec), which was about 23 percent higher than the ebb flow of 2,400 m^3/sec (84,800 ft^3/sec). Normal flood tides last approximately 5 hr and 40 min, while normal ebb tides last about 6 hr and 40 min (USAEWES 1993).

Results of the 1994 study indicated that the peak flood flow was approximately 2.4 m/sec at the inlet entrance. Current distribution also showed a shift of the flood current to east of the center of the inlet. This shift is thought to result from the filling of the scour hole and tightening of the east jetty. Tidal prisms computed from the 1992 and 1993 surveys are shown in Table 8 (USAEWES 1995).

Table 8
Shinnecock Inlet Tidal Prism (USAEWES 1995)

Survey	Tide Range[1] m (ft)	Tidal Prism[2] x 10^7 m^3 (x 10^8 ft^3)
21-23 July 1992	0.76 (2.5)	2.43 (8.58)
15 September 1993	1.22 (4.0)	3.85 (13.6)
20-21 July 1994	1.16 (3.8)	3.31 (11.7)

[1]Tide ranges determined from low water slack to high water slack based on NOAA tide tables.
[2]Tidal prisms calculated based on the flooding phase of current measurements.

Winds

Wind velocities, duration, and direction determine the characteristics of waves likely to be experienced in the study area. Wind-generated waves are the primary natural force which shapes the ocean shoreline along the southern Long Island shore. The design height of most shore structures is dictated to a great degree by the height of such waves.

Wind records from the U.S. Coast Guard and Suffolk County Highway Department for the south shore of Long Island for the period 1940 to 1959 were used to compile the percent-occurrence of winds, by direction from which winds blew, as tabulated in Table 9. Wind data were collected at the U.S. Coast Guard Stations at Tiana and Shinnecock and at the Suffolk County Highway Department gauge at Westhampton Beach. The predominant onshore winds are from the southwest. Winds from the eastern and southern quadrants, although infrequent, have an appreciable influence on the south shore of Long Island due to the essentially unlimited open ocean fetch length over which they are generated (USAE District, New York 1988).

Evaluation of wind records from the U.S. Navy Hydrographic Office for the area near the shore of Long Island indicate that winds from the westerly quadrants prevail, which is in agreement with Table 9. Monthly cumulative average winds over the North Atlantic indicate that the predominant direction of offshore surface winds is from the northwest for the period October to April and from the southwest for the period May through September (USAE District, New York 1988).

Over 50 percent of the winds which exceeded 17 m/s (33 knots) are from the west and northwest. Approximately 20 percent of the winds from the southeast quadrant exceeded 17 m/sec (33 knots). Wind data extracted in the Wave Information Study (WIS) hindcast (Brooks and Brandon 1995), while not directly applicable to the study area, represent the offshore wind environment and indicate that the predominate wind speeds range from 2.5 to 12.5 m/sec (4.8 to 24 knots), totaling about 90 percent of all recorded wind speeds. Approximately 70 percent of all recorded wind records are less than 7.4 m/sec (14.3 knots) (Moffatt & Nichol 1996).

Table 9 Estimated Annual Average Winds (1940-1959)	
Direction	**Percent of Time**
N	10
NE	9
E	9
SE	6
S	9
SW	22
W	17
NW	17
Calm	1

Waves

Waves that occur in the study area consist of sea- and swell-type waves. Locally generated waves, generally referred to as seas, are typically observed as traveling with the wind. Swells are waves generated from distant storms that enter the study area independent of the local wind conditions.

Visual surf observations were collected from the western end of Jones Beach (97 km southwest of Shinnecock Inlet) for the period October 1954 to December 1957 under a cooperative surf observation program between the U.S. Coast Guard and the Beach Erosion Board. Results show that 98 percent of the waves were from the southern quadrant and the remaining 2 percent are from the east. The waves from the southeast and southwest predominated, with 41 and 40 percent of all waves coming from these directions, respectively. During the period of observation, only 5 percent of the waves had a height of 1.2 m or greater (USAE District, New York 1988).

Wave data statistics were obtained from the WIS Atlantic Update - 1976 to 1993 (with hurricanes) for Station 78 approximately 10.5 km south of Shinnecock Inlet at a depth of 27 m (Brooks and Brandon 1995). The data consist of percent occurrence of significant wave heights and period ranges in 30-deg angles of approach increments over an 18-year hindcast period (for both storm and nonstorm conditions). Approximately 37 percent of the waves approach from azimuths (56 to 146 deg) east of shore normal (158 deg). Thirty-two percent of the waves approach from the western azimuths (169 to 259 deg). The predominant (15.3 percent) period is between 7.0 and 7.9 sec, though all of the wave period bands between the 4.0- to 4.9-sec and 9.0- to 9.9-sec bands have occurrences between 10.0 and 12.8 percent. The predominant (35.3 percent) significant wave height (in 27 m of water) is between 0.5 and 1.0 m (Brooks and Brandon 1995).

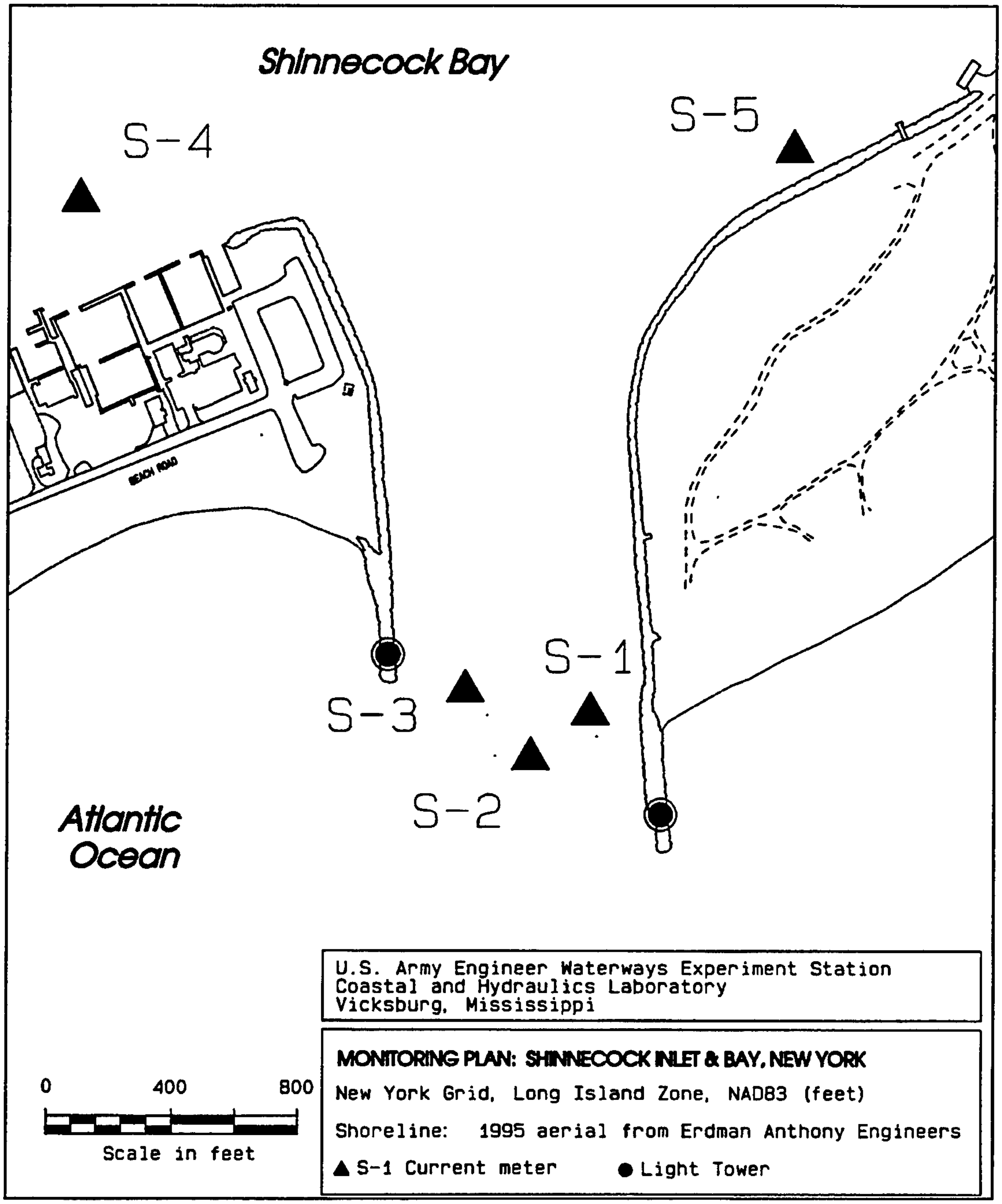

Figure 16. Current measuring stations at Shinnecock Inlet (USAEWES 1991)

4 Design Criteria

Objective

There are two primary indicators of the need for sand bypassing at an inlet: Navigational problems caused by channel shoals and downdrift beach erosion caused by trapping of littoral transport. Engineer Manual 1110-2-1616 (HQUSACE 1991) cautions against trying to design a dual-purpose bypassing system (one that tries to reduce navigation shoaling and alleviate beach changes at the same time). Although channel shoaling and downdrift beach erosion are often related at a particular site, attempting to solve both simultaneously can be difficult because:

a. The interrelationship between the two problems is often more complex than it appears.

b. The optimum approach to solving one of the problems can be very different from the optimum approach to solving the other problem. These differences can result in a compromise design that solves neither problem very well.

At Shinnecock Inlet, navigation channel shoaling has been a concern, except during the mid-1980s when both the navigation channel and ebb shoal experienced material losses. Recently, greater emphasis has been placed on downdrift beach erosion because of the historically receding shoreline, history of barrier breaches, and potential recurring damage to Dune Road. The two primary areas of concern include the beach just west of the west jetty and farther downshore, beyond 2,000 m.

The erosion hotspot directly west of the west jetty appears to be a localized problem caused primarily by a cutoff of easterly transport by the growth and attachment of the ebb shoal to the downdrift beach. The regional net sediment transport is east to west such that the natural sand bypassing mechanism is from the ebb shoal towards the west. Occasionally, transport reversals occur due to seasonal changes in wave climate, currents, or winds, causing temporary longshore transport to the east. However, as the ebb shoal has developed and attached to the downdrift shore, this easterly directed transport has been unable to reach the areas nearest the west jetty, thus exacerbating the local erosion problem (Moffatt & Nichol 1996).

The larger regional problem of reestablishing an uninterrupted sediment supply to the downdrift beaches near Tiana Beach (i.e., beyond the ebb shoal reattachment) is the objective of this effort. The problem is best addressed by a bypassing/sand management plan.

Bypassing Concepts and Methods

Sand bypassing can be described by the mode of operation and operation schedule. The mode of operation deals with the fundamental way in which the sediment is collected. In an interception mode, bypassing is performed from a location that has a readily available source of material and minimal storage volume. It only functions when sediment is moving to it and thus is best used when drift volumes and transport paths can be predicted with reasonable accuracy and the range of transport rates is not broad. An interception-mode system generally only captures a portion of the littoral drift, and it may be impractical to design for maximum transport rates associated with extreme events. Therefore, some material may be missed for bypassing because of the lack of storage. The alternative to an interception-mode system is one that has a storage area. Storage areas may be either natural or man-made such as accretion fillets, bars, deposition basins, or channels. Storage areas allow flexibility in dealing with high transport rates by trapping material that can be bypassed during times of below-normal transport. This mode allows for more scheduled operations and may be able to use a lower capacity system on a more continuous basis (HQUSACE 1991).

Operation schedule can be either continuous or periodic. A continuous system runs on a regular basis that may coincide with some convenient working period (regular working hours, daylight, etc.). Continuous operation may be used for both interception and storage modes, though during periods of high transport rates, a continuous system in interception mode may be overwhelmed. The best application for continuous operation is in the storage mode. In this mode, the system operating schedule is less affected by short-term transport rate variations because of the flexibility provided by the storage area. A system operating periodically only bypasses sand when necessary--whether by availability of sand or due to social, recreational, or environmental factors. For example, an inlet where periodic maintenance dredging removes material from the channel and places it on downdrift beaches is a periodic bypass system operating in storage mode. It operates periodically when the storage area, the navigation channel, is "full" (HQUSACE 1991).

From a conceptual view, the most desirable bypass system for Shinnecock Inlet is one that operates either continuously or periodically in a storage mode. Much of the littoral transport material at Shinnecock is trapped or stored in the large updrift fillet and ebb shoal, thereby providing ideal sources of material for bypassing. Continuous (or semi-continuous) or periodic scheduling is a factor of the system selected, which depends on both cost and how the material must be delivered to the downdrift beach. If a more or less continuous delivery schedule throughout the year is desired, then some type of fixed plant operating from the updrift fillet would be selected. If a continuous delivery is not necessarily required, costs and sand sources may allow for a floating plant operating periodically from either the channel, ebb, or flood shoal

may allow for a floating plant operating periodically from either the channel, ebb, or flood shoal or the updrift fillet.

The most significant factor in selecting a system for bypassing sand is its degree of mobility. Mobility reflects the ability of the "plant" to reach different areas of the site as well as react to changing conditions. The three classifications of bypass systems based on mobility are: fixed, mobile, semifixed (or semi-mobile). Each system and its respective components are described in the following paragraphs.

Fixed systems

A fixed system is a stationary dredging system that has been designed and built for a specific location. Fixed systems require exacting predictions on littoral transport vectors, transport pathways, and deposition patterns because they are characterized by having the entire plant fixed to one location. Examples include dredge pumps operating on/near the beach or jetty or fixed plants using jet pumps like the innovative facility at the Nerang River entrance in Australia. Fixed system components typically include a suction boom, pumps, discharge line, pump motor, and housing structure. Suction booms contain the open end of the pipe through which sand enters the bypass plant. They are generally constructed out of steel pipe and flexible hose held by a hoist and may also contain fluidizer jets to fluidize sand for pumping. Pumps are usually traditional dredge pumps driven by a motor (diesel or electric) and located in a housing nearby. The discharge line is similar to that used in other dredging systems and can be made of rigid steel or high-density polyethylene (HDPE). An example of one type of fixed plant with the pump housing located on the updrift jetty is shown in Figure 17 (HQUSACE 1991).

A jet pump could be used with fixed, semifixed, or mobile systems. Jet pumps (also known as eductors) are hydraulically powered pumps with no moving parts that rely on the exchange of momentum to do work. A simple jet pump consists of a reducing nozzle in a section of pipe followed by a mixing chamber with a suction opening and a diffuser (Figure 18). A stream of high-velocity clear water from a supply pump is forced through the nozzle. On exiting the nozzle, the high-velocity jet entrains the surrounding fluid and forces the mixture through the mixing chamber into the diffuser, where velocities slow and pressure energy is recovered. This movement of the fluid into the diffuser creates a negative pressure, inducing flow into the suction opening. When the suction opening is buried in the sand, a slurry of sand and water will be drawn into the jet pump (HQUSACE 1991).

Jet pump systems are ideally suited for areas where continuous bypassing is needed because they operate at relatively low pumping rates. However, debris can cause particular problems for jet pumps because of the small diameters. The occurrence of debris at a location will be a major factor in the overall success of jet pumps. Similarly, the presence of cohesive material will severely limit the operation of a jet pump system because of the inability to fluidize the material. Finally, a clear water intake is required nearby to provide water to drive the jet pump. This water should be taken from a sheltered region with an absence of shoaling (HQUSACE 1991).

One example of a fixed system using jet pumps is the Nerang River Entrance Bypass Plant in Australia. This plant uses 10 jet pumps located on a trestle that extends out into the surf. Each jet pump can be operated individually or in combination with other units. A system schematic is shown in Figure 19.

Submersible pumps may be considered as an alternative to jet pumps in certain situations. These pumps, electrically or hydraulically driven, are lowered directly into the material to be moved. They are relatively small and can be deployed with a minimum of support equipment, and they are less immune to plugging than jet pumps. However, they contain more moving parts and thus require more servicing than jet pumps (HQUSACE 1991).

Mobile systems

Mobile systems are designed such that the plant can be moved to different areas within the project site or to other projects. Examples include floating plant (i.e., dredges), movable jet pumps, or land-based dredges (e.g., *Crawlcat*). The majority of mobile bypass systems use conventional floating dredges (hopper dredges, cutterhead dredges, etc.), which can either be purchased or contracted from dredging companies. Contracting is the usual procedure because of the flexibility of specifying the dredging requirements (volume, location of material, distance to pump, etc.) so that dredging companies can bid based on the available equipment. If a dredge is purchased, the flexibility of dredging a larger volume or pumping a greater distance may be lost because of the permanency of plant ownership (HQUSACE 1991).

Mobilization and demobilization (mob/demob) is a factor to be addressed when considering using a dredge for bypassing. Mob/demob involves moving the equipment to the project site, transport, and placement of pipe and other equipment (crew boats, tugs, and/or barges). The costs of mob/demob can be a significant portion of overall project costs, with some estimates varying between 10 and 40 percent of total costs (HQUSACE 1991).

Bypass frequency should also be considered when evaluating a mobile system. Quantities large enough to require yearly bypassing may make the purchase of a dredge more feasible, though the mob/demob costs would be repeated every year. By dredging larger volumes every 2 or 3 years, mob/demob costs can be amortized, and depending on the operational costs, annual dredging costs may be reduced.

Mobile systems sometimes use deposition basins to accumulate littoral material for bypassing at a later time. Some deposition basins are linked to a weir jetty, which allows littoral transport to move along the beach and into the basin located inside the inlet. This material can then be periodically dredged and the material placed downdrift. Figure 20 shows a typical weir jetty bypass system. Deposition basins may also be located in the inlet throat so material from the gross transport (i.e., from both directions) can be collected. As the basin fills, periodic dredging can remove the material in a manner similar to traditional channel maintenance (HQUSACE 1991).

Semifixed (semi-mobile) systems

A semifixed plant is one that is generally fixed to a project site but not to a specific area within the site. The mobility afforded by a semifixed plant increases the effective storage area available to be mined. An example of this is the Indian River Inlet, Delaware Bypass System (Figure 21). This plant is fixed in that the dredge pumps are immovable on the backbeach, but the jet pumps are deployed from a crane that can mine sand from the entire jetty fillet, thus maximizing the storage area. During storms, the crane can move near the pump house on the backbeach to reduce potential storm damage.

A list of equipment that can be or has been used in fixed, mobile, and semifixed bypass systems is shown in Table 10 (HQUSACE 1991). This list is not exhaustive and could be expanded based on particular needs or sites.

Table 10
Equipment That Can Be Used for Bypassing

Floating Dredges	Land-based	Hydraulic Equipment
Trailing suction hopper	Dragline	Dredge pump
Cutter-head	Clamshell	Jet pump
Plain suction	Backhoe	Other solids-handling pumps
Bucket ladder	Bulldozer	
Clamshell	Dump truck	
Dipper		
Backhoe		

Design Considerations

Quantity

In spring/summer 1996, the Baltimore office of MNE, under contract to NAN, conducted a detailed coastal processes study of Shinnecock Inlet and the adjacent shoreline. One aspect of the study was to revisit and improve upon previous sediment budget estimates for this reach of Long Island. USAE District, New York (1988) summarized the sediment budget at Shinnecock Inlet as having an approximate yearly net transport of 229,000 m^3 (300,000 yd^3) toward the west with about 76,500 m^3/year (100,000 yd^3/year) being deposited on the ebb shoal. The NAN and MNE considered those values preliminary because of limited data on ebb shoal growth, and MNE sought to improve the confidence in longshore transport rates by performing analysis with

up-to-date techniques. To begin, MNE divided the shore near Shinnecock Inlet into three littoral cells-east of inlet, inlet, and west of inlet. They also split their analysis into three time periods based on the different stages of inlet evolution (Table 11). MNE examined historical shoreline positions to estimate beach/dune erosion and deposition volumes for each of the littoral cells.

Table 11
Sediment Budget Epochs (after Moffatt & Nichol (1996))

Period	Years	Description
Epoch I	1830-1933	Preinlet
Epoch IIA	1938-1956	Postinlet & prejetties
Epoch IIB	1956-1984	Postjetties
Epoch III	1979-1995	Increased erosion

Using wave energy flux calculations, MNE made a thorough analysis of longshore transport rates into and out of each cell balanced by ebb and flood shoal volume changes, offshore/onshore volume changes, and channel shoaling and dredging volume changes. These results showed that the previous estimate of 229,000 m^3/year (300,000 yd^3/year) overestimated transport by over 100 percent. MNE revised sediment transport rates are summarized in Table 12.

The design bypass rate should consider both longshore transport and ebb and flood shoal and channel volume changes. Ideally, one should identify the shoal and channel volume balances (that material that is deposited and eroded) and compare these with the longshore transport rate to determine the target mechanical bypass rate. To do this, accurate hydrographic surveys of the ebb shoal, flood shoal, and channel over time must be available for analysis, along with dredging records. At Shinnecock Inlet, the only complete ebb shoal surveys were conducted recently (hydrographic and SHOALS surveys, 1996), so only a long-term average ebb shoal volume change rate can be calculated. However, this volume change rate is misleading since it cannot account for probable nonlinear volume changes as the ebb shoal has grown to its current size. Additionally, little, if any data exist for flood shoal growth since the inlet was created, which severely clouds the entire inlet sediment budget picture. Based on the volume change rates shown in Table 12 from MNE, one can see that the rate of ebb shoal growth has decreased between the three epochs presented. Initial ebb shoal growth was relatively high (117,000 m^3/year from 1938 to 1949; see "Volumetric analysis, 1933 to 1949" on page 15), but the overall average ebb shoal growth from 1933 to 1996 (110,000 m^3/year, see "Volumetric Analysis, 1933-1996" on page 16) suggests that the rate of growth decreased in later years. Probable factors contributing to the decrease in growth in later years include updrift sediment impoundment resulting after jetty construction in 1953 and 1954, natural relocation of the main ebb shoal channel in the mid-1980s, and dredging of the deposition basin in 1990 and 1993. This limited information leaves too much unknown to comfortably estimate the rate of deposition (or erosion) of the ebb and flood shoals and channel. Therefore, the design bypass rate will be based primarily on the longshore transport rate as developed by MNE with some flexibility for uncertainty.

Table 12
Revised Sediment Transport Rates (after Moffatt & Nichol (1996))

Period[1]	East directed m^3/yr (yd^3/yr)	West directed m^3/yr (yd^3/yr)	Gross m^3/yr (yd^3/yr)	Net m^3/yr (yd^3/yr)	Ebb Shoal Deposition m^3/yr (yd^3/yr)
Epoch IIA (1938-1956)					
East of inlet	90,200 (118,000)	171,000 (224,000)	261,000 (342,000)	81,000--west (106,000--west)	122,000 (159,000) ebb & flood
West of inlet	90,200 (118,000)	171,000 (224,000)	261,000 (342,000)	81,000--west (106,000--west)	
Epoch IIB (1956-1984)					
East of inlet	90,200 (118,000)	171,000 (224,000)	261,000 (342,000)	81,000--west (106,000--west)	23,000 (30,000)
West of inlet	82,600 (108,000)	147,000 (192,000)	229,000 (300,000)	64,000--west (84,000--west)	
Epoch III (1979-1995)					
East of inlet	124,000 (162,000)	180,000 (235,000)	304,000 (397,000)	56,000--west (73,000--west)	-23,000[2] (-30,000[2]) ebb & flood
West of inlet	101,000 (132,000)	199,000 (260,000)	300,000 (392,000)	98,000--west (128,000--west)	

[1] Epoch I not analyzed because marked and consistent changes in erosion and accretion patterns were not observed, and sediment budget could not be developed.
[2] Includes effect of dredging 511,000 m^3 (668,000 yd^3) in 1990 and 363,000 m^3 (475,000 yd^3) in 1993 and filling of scour hole.

Moffatt & Nichol (1996) show that during Epoch II, the net westward transport of sand was 81,000 m^3/year (106,000 yd^3/year), and during Epoch III, net westward sand transport is slightly larger, though of the same order of magnitude (98,000 m^3/year or 128,000 yd^3/year). The design bypass rate should be no less than the minimum historical quantity reported by USAE District, New York (1988) being deposited on the ebb shoal (approximately 76,500 m^3/year or 100,000 yd^3/year). However, because of the increased downdrift transport experienced in Epoch III, the target average bypass rate should be around 100,000 m^3/year (131,000 yd^3/year) with capabilities to transport up to 115,000 - 134,000 m^3/year (150,000 - 175,000 yd^3/year).

The results shown in Table 12 indicate that although the net sediment transport is to the west, there is a significant portion (30 - 40 percent) of the gross transport that moves to the east. The east-directed portion of this gross rate can be important in designing an inlet management system. Gross transport is an important factor when evaluating channel shoaling. Sediment traveling in either direction (east or west in this case) can make its way into the inlet channel, which means the gross transport rate must be considered when estimating shoaling rates and calculating dredging quantities and frequencies. The gross transport is also important when

designing a bypass system, because the breakdown between major and minor transport directions may be seasonally related. For example, if during a certain period of the year the predominant direction of transport is opposite to the net yearly direction, it may be counterproductive to operate a bypass plant during that time.

Seasonal effects

The seasonal sediment transport direction was analyzed at Shinnecock Inlet to determine if certain seasons or months of the year were associated with a predominant transport direction that differed from the annual net west-directed transport. The procedure used the monthly statistics of wave height, period, and direction from the 1976-1993 WIS (Brooks and Brandon 1995) data for Station 78 off the coast of Long Island, NY (this is the same data set used by MNE for the sediment budget calculations). Following a related procedure developed by Bodge, Creed, and Raichle (1996), the offshore wave data were easily transformed from offshore to incipient breaking. From incipient breaking, an idealized potential longshore transport rate was calculated for each month using methodology outlined in the *Shore Protection Manual* (1984) with arbitrary constants. Because of some calculation simplifications and assumptions, the actual transport quantities are less meaningful than relative monthly transport differences and directions. The analysis indicates that during June and July, the predominant direction of transport is to the east. In May and August, transport is to the west, but the relative rate is so small that the total period from May through August could be considered as months where easterly (or nonwesterly) transport predominates. Conversely, October and September generated the largest transport rates (westerly) for either direction for the entire year. December also produced easterly transport, but the rate was small compared to the west-directed transport occurring in November and January.

These results suggest that any bypass system (especially a fixed system) probably should not be operated from May through August in any year. This period also coincides with part of the summer tourist season during which beach nourishment may not be desired. If a fixed or semi-fixed plant is used, the period from November through April/May (to avoid tourists and eastward transport) would be optimal for a continuous operation during acceptable weather. Operational constraints (winter waves, etc.) may limit using a floating plant system during the August through May time period. However, if a floating plant system were used on a periodic basis between August and May, downdrift placement from March to May may be preferred over placement during September/October. Material placed in the spring would be initially exposed to an easterly transport during June and July, which would help to slow the westerly movement out of the system. If material were placed in September/October, it would be initially exposed to the largest westerly transport of the year and thus would not remain where placed as long. Moffatt & Nichol (1996) hypothesize that the area where the ebb shoal attaches to the beach on the downdrift side may serve to limit this localized eastward transport (between the western limits of the ebb shoal and the west jetty), thereby essentially storing it (west of the ebb shoal or in the ebb shoal itself) for westward transport after July. If periodic placement were conducted during September and October, the physical processes that cause the largest monthly transport

during those times would contribute to a faster movement of material through the area to the west.

Sand sources

Four areas can be considered as sources of sand for use in a bypass system. The choice of a particular source may to some extent dictate the method of bypassing. The pros and cons of each of the four locations are discussed below.

a. The **updrift fillet** contains a substantial volume of sand both subaerially and subaqueously that could be used for bypassing. By comparing the shoreline positions and nearshore slopes of the 1933 (pre-inlet) and 1995 hydrographic surveys, at least 4.1 million m^3 (5.4 million yd^3) of material can be identified in the first 1 km of updrift fillet. The design bypass quantity of 100,000 m^3/year (131,000 yd^3/year) is only 2.4 percent of the total volume in the fillet, so bypassing from this source should have minimal impact on the fillet. For design, the 100,000-m^3 (131,000-yd^3) annual volume (or part thereof if combined with other sources) could be bypassed from the fillet using either a fixed or semifixed plant on the subaerial portion of the fillet or a floating or semifixed plant on the subaqueous portion of the fillet. In the case of a fixed or semifixed plant (subaerial or subaqueous), a submerged pipeline could transport the material to the downdrift beach. Concerns associated with using the updrift fillet as a sand source may include: (1) danger of undermining the east jetty if too much material is removed from near the toe; (2) opposition by property owners to removing sand from the first 1 km of beach (even though this is County property) for fear that their fronting beaches will erode more rapidly to replace the removed material. The small percentage of material required and the rate of resupply from the east makes this concern not scientifically warranted; and (3) beach bathing safety related to equipment on the beach and any holes/craters formed.

b. Use of the **ebb shoal** as a source of sand for bypassing can be controversial. However, HQUSACE (1991) states that if the bypassing amount is a small percentage of the total ebb shoal volume, removal of this material should cause no problems. Removal of large percentages of material is believed to have significant impacts on the local coastal processes, but these impacts have not yet been adequately documented at inlets (Cialone and Stauble 1998). The 1996 SHOALS survey indicates that the ebb shoal contains approximately 6.4 million m^3 (8.4 million yd^3) of sand. The design bypass rate of 100,000 m^3/year (131,000 yd^3/year) would only account for about 1.6 percent of the total ebb shoal volume. Strategic dredging to "skim" off small volumes of material (i.e., from the seaward slope) over large areas should minimize ebb shoal and downdrift impacts (paragraph (1) below), especially if the ebb shoal is nearing equilibrium. The advantage of this location for a sand source is that material naturally bypassing the inlet comes from the ebb shoal. A bypass system that takes from the ebb shoal thus works in the same manner as nature. Using the ebb shoal as a sand source for bypassing will require a floating plant

(i.e., dredge). Cialone and Stauble (1998) document historical ebb shoal mining operations and note several issues that should be considered:

(1) Mining an ebb shoal can be a major or minor perturbation to an inlet system, depending on the volume mined, depth of mining, and location on the shoal. The impact of ebb shoal mining will to some extent be experienced by all other parts of the system.

(2) Use of an ebb shoal as a source of sand will likely cause an increased rate of downdrift erosion because any material that was formerly naturally bypassed will now go to fill in the ebb shoal. Walther and Douglas (1993) concluded that impacts would be reduced with shallow cuts.

c. The **flood shoal** is also a source of material that may be used for bypassing. McCormick (1971) determined that the Shinnecock flood shoal contained sand very similar to that found along the open coast. However, little is known about the consequences of flood shoal mining on inlet and bay circulation. Therefore, it is probably safe to assume the same recommendations from HQUSACE (1991) for ebb shoals-if the volume of material to be removed is a small percentage of the flood shoal volume, no adverse consequences would be expected. Similar to ebb shoal mining, a floating plant will likely be required, even if it has significant exposed dry beach, because of its isolation from stable land. Concerns related to mining the flood shoal include effects to the tidal prism and possible wetland habitat impacts.

d. The **navigation channel** may be considered as a source of material for bypassing, especially if periodic maintenance dredging is required. In many instances, a typical maintenance dredging plan calls for removal of channel material, which is then placed at a downdrift beach. This is essentially "sand bypassing" by another name. If shoaling quantities are sufficient in both volume and time, a sand bypass system may be merely a specific plan to place channel maintenance material at the appropriate place on downdrift beaches.

If required bypass quantities are too large for any one of the sources listed above, then a combination of sources may be used. For example, a specified quantity may be removed from the subaqueous updrift fillet, navigation channel, and ebb shoal by floating plant that together would minimize specific source impacts, yet still achieve the desired total quantity. For example, at Indian River Inlet, the fixed bypassing has been supplemented by flood shoal mining (Clausner et al. 1991).

Placement options

The area west of Shinnecock has two problem erosion areas that would benefit from beach nourishment activity. As mentioned previously, the erosion hotspot directly adjacent to the west

jetty is a localized phenomenon. This phenomenon is probably caused by the cutoff of waves and east-directed sand transport due to the attachment of the ebb shoal to the beach as well as the cutoff of westerly transport by the main body of the ebb shoal and the navigation channel. Though beach nourishment is needed at this location, bypassing material updrift of this site is not the ideal solution. Because the local transport between the ebb shoal and the west jetty is predominantly east-directed and there is no physical constraint to hold material next to the jetty, material bypassed would likely continue the current trend of traveling either through or around the jetty to the inlet channel or offshore. This approach would, therefore, fail to restore the regional westward transport, continue to starve the downdrift beaches, and possibly may remove the bypassed material from the littoral system. Other solutions, as proposed by Moffatt & Nichol (1996) (e.g., groins in conjunction with beachfill, T-head groins, etc.), are more suitable for this erosion hotspot.

The other placement options are either immediately west of the ebb shoal or directly at the eroding downdrift beaches at Tiana Beach (see page 12, "Supplemental Documentation," in USAE District, New York (1988) and Moffatt & Nichol (1996) shoreline change rates). Placing the material just west of the shoal would be more cost-effective because of the shorter haul or pump distance (2.4 km versus 4.8 km to Tiana Beach). Natural processes would then transport material westward as part of the natural littoral transport. Placement at Tiana Beach would immediately address the need of local residents for sand, though it may not solve their problem if more material is needed than is mechanically bypassed. The Tiana Beach location would be more expensive due to the longer haul or pump distance and may even preclude the use of a fixed plant because of the need for booster pumps.

There are two primary methods of placement for a bypass system: nearshore and onshore (beach nourishment). These options depend on the method of bypassing used, so they will be discussed in general here and elaborated on in the **Alternative Analysis** section. Of the two methods, placing material directly on the beach is preferred because it accomplishes the desired result (widening and/or raising the dry beach). Nearshore placement is a viable alternative; however, design and construction of these features in shallow water is complex, and though the littoral system would be nourished, onshore movement of material has not been evaluated. Also, public acceptance of nearshore placement is usually marginal at best. Construction costs may also become a factor in selecting one method over another. If bypassing is being conducted by floating plant, nearshore placement may be preferred because of the relative ease of placement from hopper dredges. Onshore placement with a hopper dredge is common through an offshore buoy connection to a submerged pipe, but increased costs may be expected due to the time required to pump out material. A fixed bypass plant at the updrift fillet would probably place material directly on the beach through a pipeline. However, problems with the pipeline crossing under the inlet and running along (or buried under) the beach can be expected. Onshore placement may also require earth-moving equipment to redistribute material as it is discharged from the pipe, similar to traditional beach nourishment projects (though at a reduced level due to the smaller volumes).

Constraints

Conditions specific to Shinnecock Inlet and the area may limit or prohibit the use of a particular bypass system. In some cases, the preferred system design can work within these constraints, or the constraint can be modified so as to minimize or eliminate problems. At Shinnecock Inlet, one limitation that must be considered is the thickness of the sand layer. This thickness is important for two reasons: determining the maximum depth of dredging or sediment removal at the updrift fillet and in the ebb shoal (particularly important for the *Punaise* system); and determining the conditions for tunneling under the inlet (or dredging a transverse channel across the inlet) for laying a pipe. Five cores (one 12.2-m (40-ft) and four 6.1-m (20-ft)) were taken at Shinnecock just offshore of the updrift fillet in November 1996. These cores were taken in approximately 6-m water depths and showed an average sand layer thickness of about 6 m. Below the 12.5-m depth (sand layer thickness of 6.5 m) the cores contained mud to the bottom of the core (Alpine Ocean Seismic Survey, Inc. 1997).

A second factor which constrains a fixed or semifixed alternative is the size of the ebb shoal. As mentioned previously, the Shinnecock Inlet ebb shoal extends 2.4 km beyond the west jetty (see Figure 14 for extent of ebb shoal). The desire to maintain the regional littoral transport will require that any bypass material must be placed or pumped at least to this point to avoid the ebb shoal shadow zone. This constraint may require the use of booster pumps with any alternative having a pumping component (i.e., fixed/semifixed, *Crawlcat/Crawldog*, or *Punaise*).

Equilibrium Ebb Shoal Volume

Walton and Adams (1976) made a study of 44 inlets from the Pacific, Atlantic, and Gulf coasts of the United States to evaluate the relationships between ebb shoal volume and tidal prism. Their study divided the 44 inlets into three groups: highly exposed, moderately exposed, and mildly exposed coasts. All of the highly exposed inlets were located on the Pacific, while the moderately exposed inlets were primarily located on the Atlantic coast and the mildly exposed inlets were on the Atlantic and Gulf coasts. They generally concluded that more sand is stored in the ebb shoal at inlets on a low-energy coast than on a higher-energy coast.

The relationships that Walton and Adams (1976) devised are based on the following equation in English units:

$$V = a P^{b} \tag{2}$$

where

V = volume of sand stored in the ebb shoal in (yd^3)
P = tidal prism in (ft^3)
a,b = correlation constants

Linear regression was used to find b for each of the groups and for all inlets combined. Because there was no significant difference in the exponential correlation coefficients, the value of b was set to 1.23 (corresponding to the highly exposed coast) primarily because there was a minimum of scatter for the Pacific coast inlets. Using $b = 1.23$, the correlation coefficient a was determined for each group based on respective inlet prism - ebb shoal volume relationships. The resulting equations for each condition are:

$$V = 8.7 \times 10^{-5} P^{1.23} \text{ (highly exposed coasts)} \tag{3}$$

$$V = 10.5 \times 10^{-5} P^{1.23} \text{ (moderately exposed coasts)} \tag{4}$$

$$V = 13.8 \times 10^{-5} P^{1.23} \text{ (mildly exposed coasts)} \tag{5}$$

$$V = 10.7 \times 10^{-5} P^{1.23} \text{ (all inlets)} \tag{6}$$

Though Shinnecock Inlet is located on the somewhat more active northern mid-Atlantic coast, it is oriented in a generally east-west direction, thus limiting its open ocean exposure. To determine the stage of equilibrium development of the ebb shoal at Shinnecock Inlet, either the moderately exposed or mildly exposed relationship should be most appropriate. Tidal current measurements and calculations determined that the normal tidal prism for Shinnecock Inlet was 2.43×10^7 m^3 (see Table 8). A probable maximum equilibrium ebb shoal volume can be estimated by using the tidal prism from a spring tide condition (i.e., 3.31×10^7 m^3 or 3.85×10^7 m^3). Results from each of these calculations are given in Table 13.

As discussed previously, the measured volume of sand in the ebb shoal is 6.4 million m^3 (8.4 million yd^3). According to the Walton and Adams (1976) relationships, the equilibrium volume for the Shinnecock Inlet ebb shoal should be between 7.8 and 10.3 million m^3 (10.2 and 13.4 million yd^3), which means currently it has between 63 and 82 percent of its equilibrium volume. A theoretical volumetric limit to the equilibrium volume can be predicted by examining the spring tide equilibrium volumes, which range between 11.5 million and 18.1 million m^3 (15.0 million and 23.7 million yd^3), depending on the exposure of the coastline.

Table 13
Equilibrium Ebb Shoal Volume Calculations[1]

Tidal Prism[2]	Ebb Shoal Volume, yd^3 (m^3)		
	Moderately Exposed	Mildly Exposed	All
Normal (8.57×10^8 ft^3)	10.2×10^6 (7.8×10^6)	13.4×10^6 (1.03×10^7)	10.4×10^6 (7.95×10^6)
Spring Tide in 1993 (13.6×10^8 ft^3)	18.0×10^6 (1.38×10^7)	23.7×10^6 (1.81×10^7)	18.4×10^6 (1.41×10^7)
Spring Tide in 1994 (11.7×10^8 ft^3)	15.0×10^6 (1.15×10^7)	19.7×10^6 (1.51×10^7)	15.2×10^6 (1.16×10^7)

[1] All calculations were made with English units and converted to SI.

[2] USAEWES (1995) and Table 8.

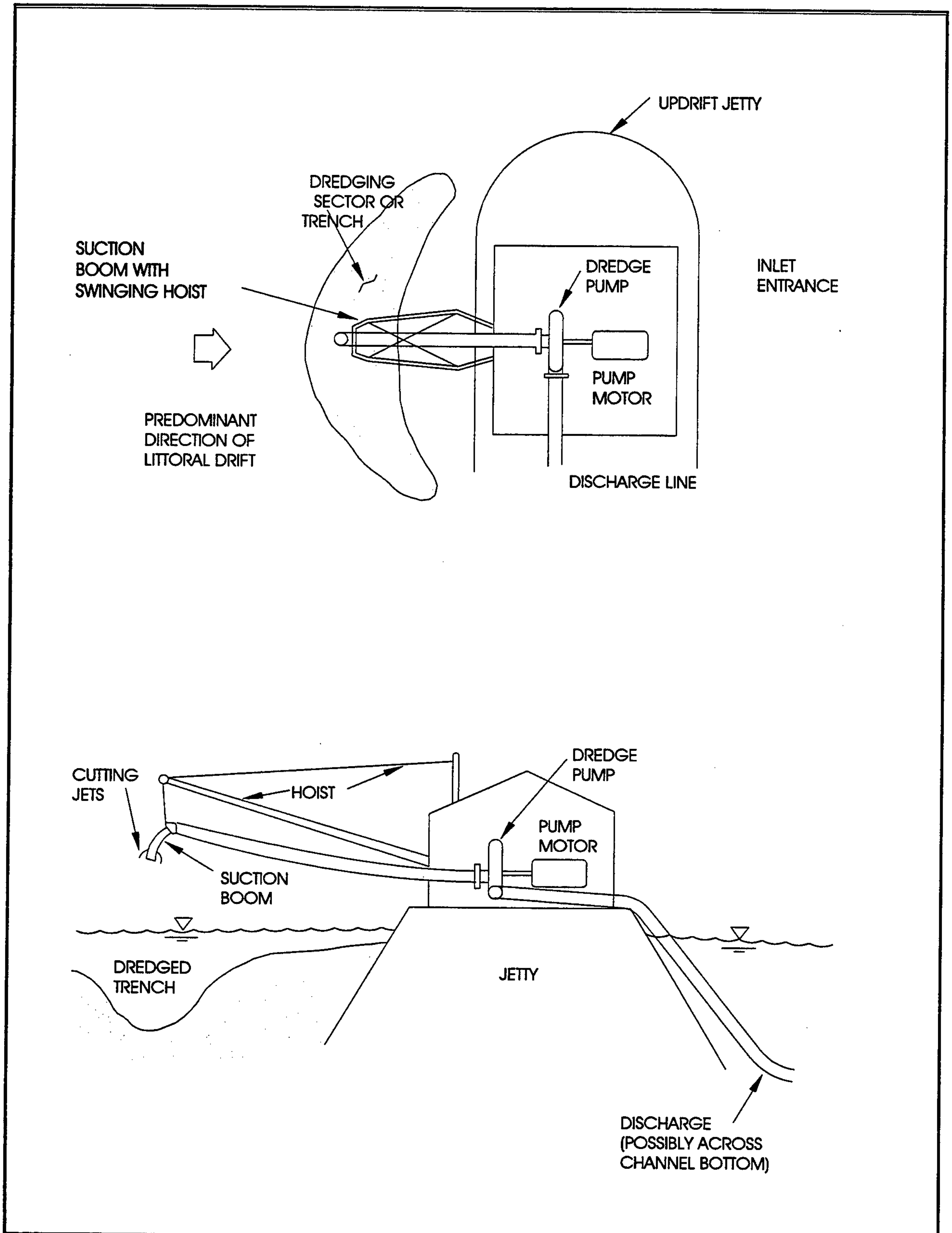

Figure 17. Fixed plant bypass system with pump house located on updrift jetty (HQUSACE 1991)

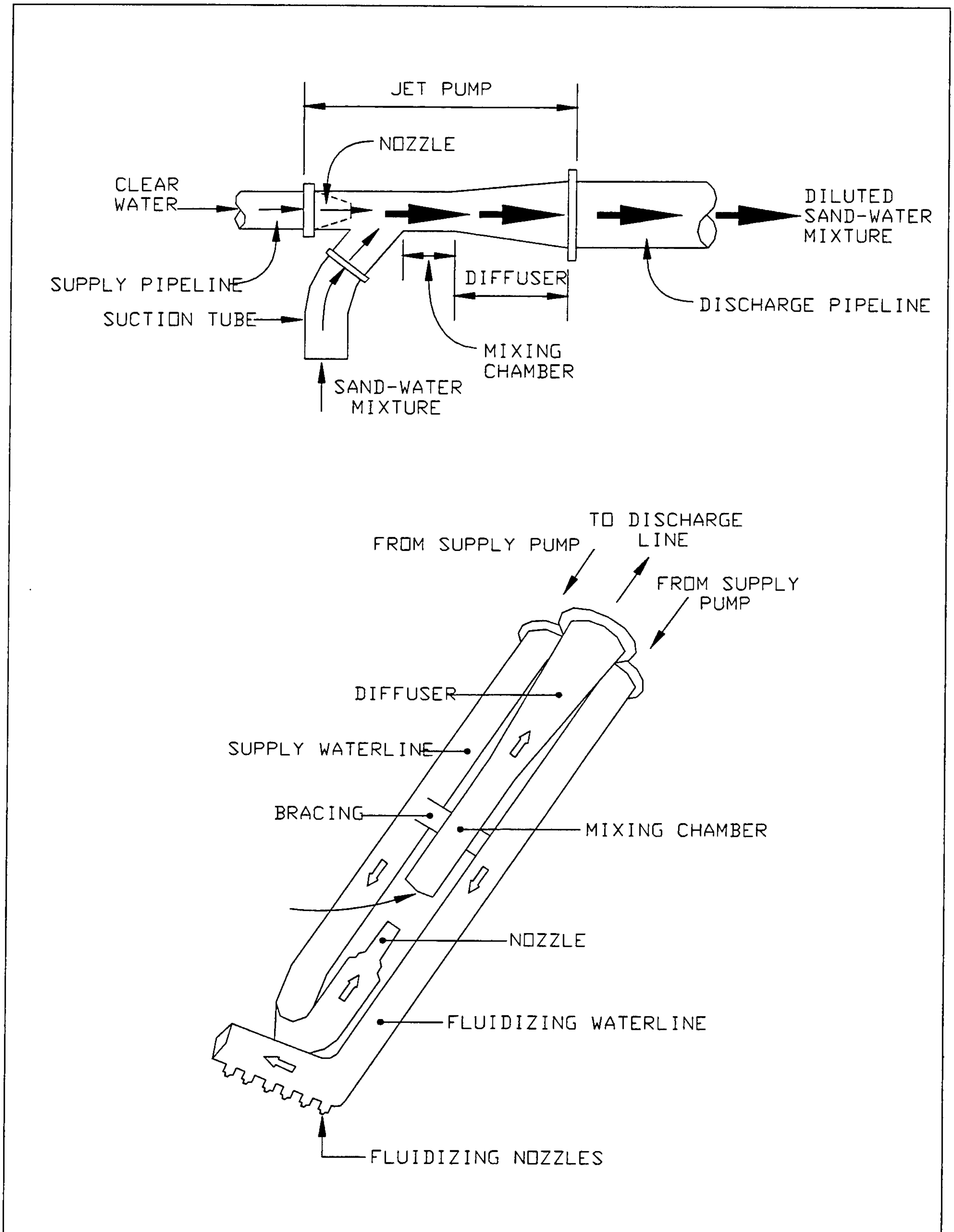

Figure 18. Jet pump

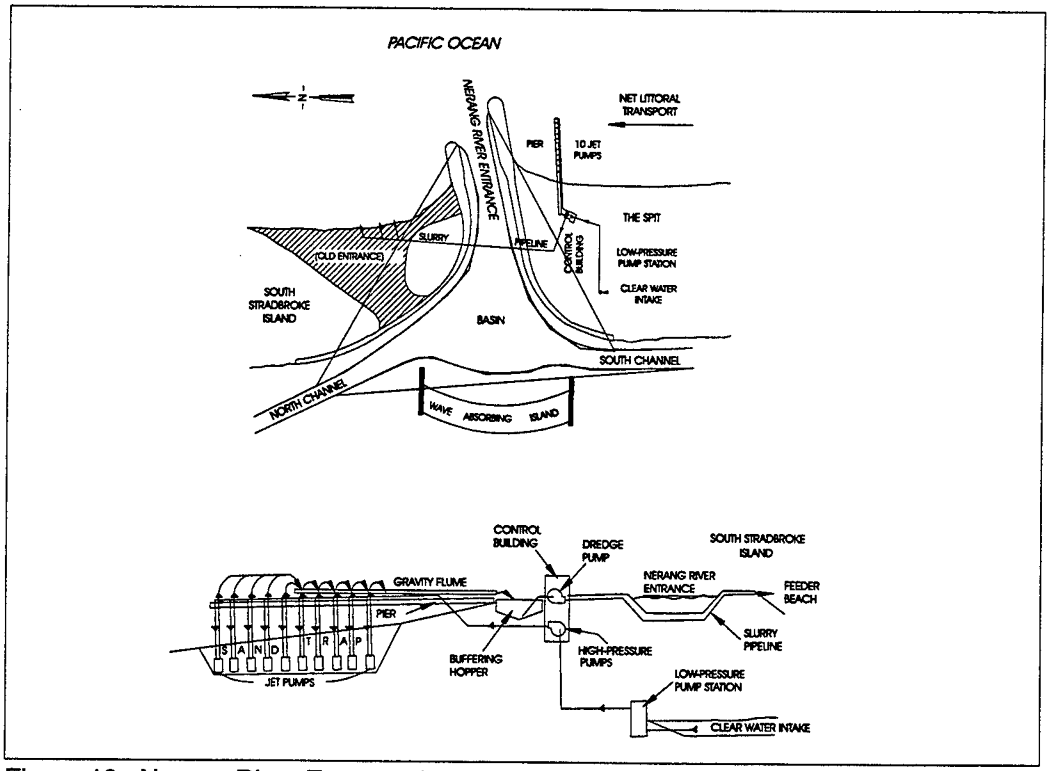

Figure 19. Nerang River Entrance bypass system

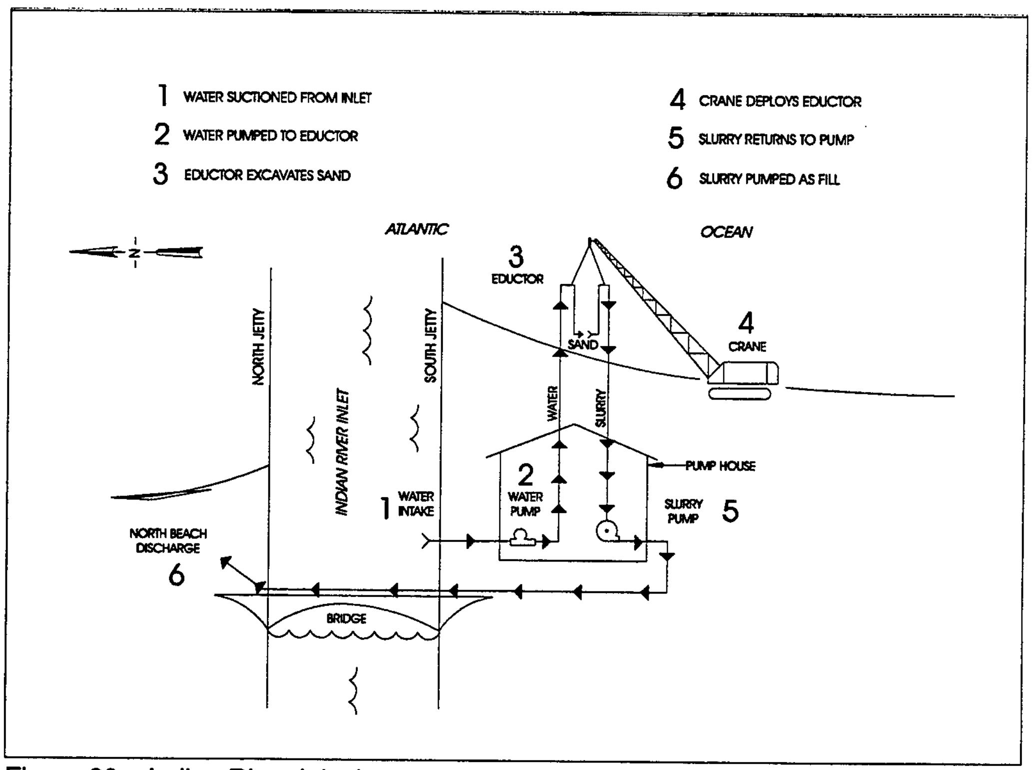

Figure 20. Indian River Inlet bypass system

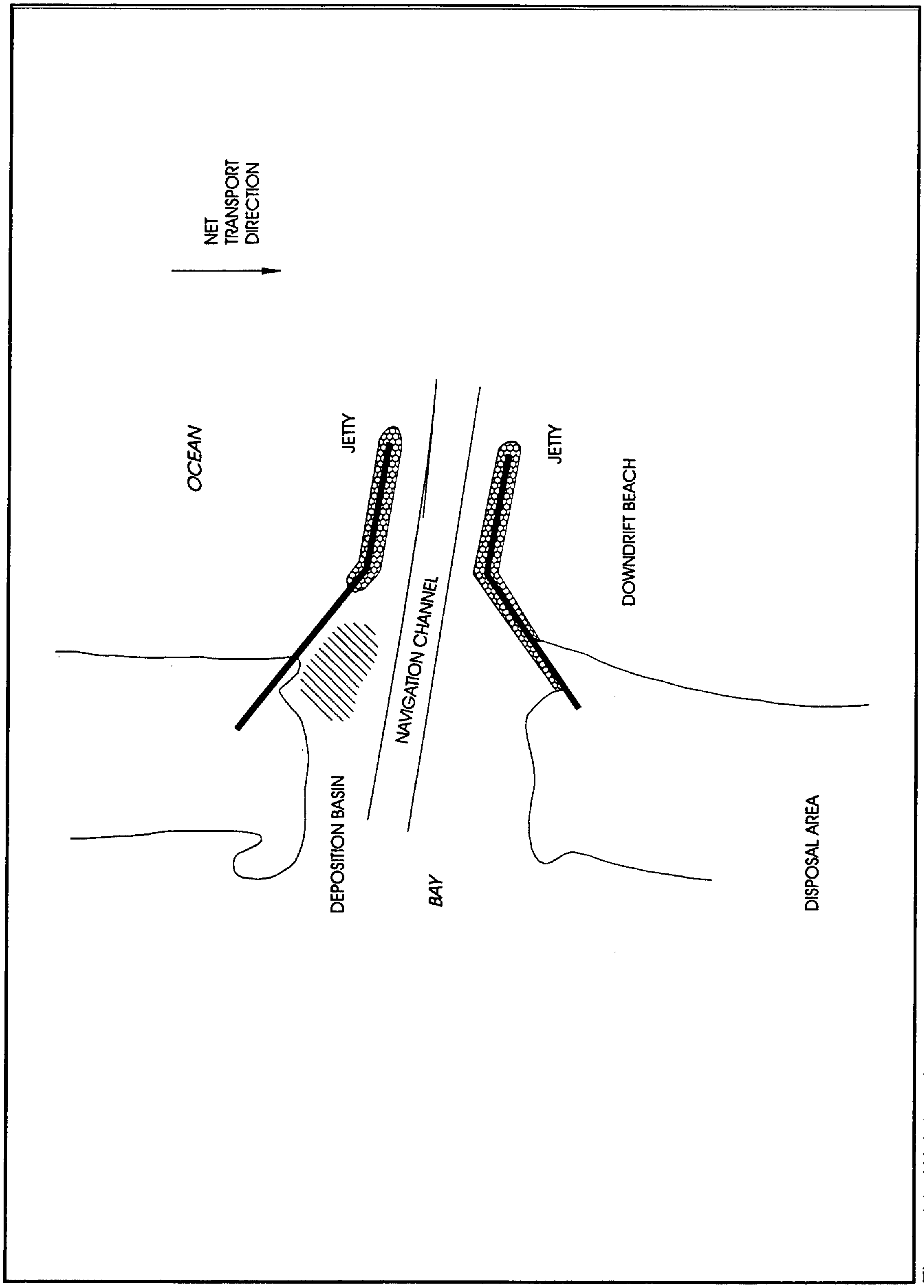

Figure 21. Weir jetty bypass system

5 Bypassing Alternatives

Various alternatives exist for bypassing sand around Shinnecock Inlet. Of the general descriptions discussed previously, three will be expanded, and a new and old technology will also be discussed.

Alternative 1: Floating Plant

The first alternative is the floating plant (dredge), which is the most traditional method of bypassing because of its use in maintenance dredging and availability on contract (as opposed to capital outlay needed to purchase fixed equipment). This alternative analysis only considers a trailing suction hopper dredge, though other floating plants may be feasible. The hopper dredge is more maneuverable than other dredges and can more easily operate in an open ocean environment. A trailing suction hopper dredge can also efficiently remove thin layers of sand over large areas, which is a desired characteristic for using the updrift fillet, ebb shoal, and flood shoal as sources of sand. By using all three areas, the impact of bypassing 100,000 m^3/year (131,000 yd^3/year) (or 200,000 (262,000 yd^3) or 300,000 m^3 (393,000 yd^3) on a 2- to 3-year cycle) on any one source location will be minimized. For example, the dredge could remove a portion of the required volume from each of the areas (offshore areas of the fillet, seaward slope of the ebb shoal, and remote locations of the flood shoal) such that sediment removal will be less likely to adversely affect local transport rates and inlet hydraulics. Figure 22 is a plan-view of potential dredging (flood shoal borrow area not shown due to lack of data to characterize flood shoal) and placement locations.

As was mentioned previously, performing bypassing during longer dredging cycles (2 to 3 years as opposed to every year) may help to keep total costs down by spreading the mob/demob costs over several years. However, with proper planning and contracting, an annual dredging cycle may be able to reduce remob/demob costs by storing equipment and pipes in the vicinity (Great Lakes Dredge & Dock 1996).[1] Cost estimates are presented for dredging cycles of 1, 2,

[1]Personal communication with Mr. Bill Murchison of Great Lakes Dredge & Dock, Chicago, IL.

and 3 years (100,000 m^3, 200,000 m^3, and 300,000 m^3, respectively), including different placement options (onshore versus nearshore).

According to Great Lakes Dredge & Dock (1996), mob/demob costs for a hopper dredge operating at Shinnecock Inlet with an approximate 4.8 km (3-mile) sail distance to Tiana Beach would be between \$500,000 and \$750,000. Estimated unit costs for dredging and placing material vary between \$6.21/m^3 (\$4.75/yd^3) for beach placement using a pump-out buoy and \$3.92/m^3 (\$3.00/yd^3) for nearshore placement. Including contingencies of 10 percent increases the cost to \$6.83/m^3 and \$4.31/m^3 (\$5.23/yd^3 and \$3.30/yd^3), respectively. Additional increases for engineering and design (E&D at 15 percent) and supervision and administration (S&A at 10 percent) give final unit costs of \$8.54/m^3 and \$5.39/m^3 (\$6.54/yd^3 and \$4.13/yd^3), respectively. Estimated costs are summarized in Table 14.

Table 14
Cost Summary for Alternative 1: Floating Plant

Cycle/ Placement	Quantity m^3(yd^3)	Mob/Demob[1] (\$)	Dredge/Place \$/m^3(\$/yd^3)	Annual Cost[2] (\$)	Annual Unit Cost[3] \$/m^3 (\$/yd^3)
1 Year					
Nearshore	100,000 (131,000)	500 K	5.39 (4.13)	1.039 M	10.39 (7.93)
Onshore	100,000 (131,000)	500 K	8.54 (6.54)	1.354 M	13.54 (10.34)
2 Years					
Nearshore	200,000 (262,000)	750 K	5.39 (4.13)	866 K	8.66 (6.61)
Onshore	200,000 (262,000)	750 K	5.39 (6.54)	1.165 M	11.65 (8.89)
3 Years					
Nearshore	300,000 (393,000)	750 K	5.39 (4.13)	712 K	7.12 (5.44)
Onshore	300,000 (393,000)	750 K	8.54 (6.54)	996 K	9.96 (7.60)

[1] Mob/Demob and unit costs provided by Great Lakes Dredge and Dock, Chicago, IL. Mob/Demob costs less for 1-year cycle based on assumption of on-site storage of some equipment to lower mobilization.
[2] Annual costs are calculated using the project cost amortized over the project cycle using 7-3/8 percent interest rate (as recommended by NAN) (see Appendix E).
[3] Annual cost divided by 100,000 m^3 (for 131,000 yd^3).

Alternative 2: Semifixed Plant

The Indian River Inlet Bypass Plant is used as a reference for the second alternative. At Indian River Inlet, a crawler crane positions a jet pump on the updrift fillet near mlw. The jet

pump is supplied motive water by a 253-kW (340-HP) pump which takes clear water from the inlet. The jet pump discharge supplies a 246-kW (330-HP) booster pump, which then pumps the slurry to the downdrift beach (approximately 457 m with capability to pump as far as 1,070 m). Both pumps and the diesel engines that drive them are located in a pump house well back from the open beach. The Indian River Inlet Bypass Plant is used as a guide for the Shinnecock analysis, because it has a similar design bypass rate (on the order of 76,500 m^3 (100,000 yd^3/year)) and because this type of semifixed plant provides greater flexibility in reaching a larger area of the updrift fillet and can be better protected during storms by moving the crane and jet pump assembly to the backbeach.

Several factors that must be considered in analyzing this type of bypass plant include:

a. Depth of jet pump operation.

b. Proximity to local structures (east jetty at Shinnecock).

c. Pipe discharge location (pipe lengths, booster pump requirements, etc.).

d. Operating time.

e. System capacity.

f. System element costs.

The depth below ambient bottom at which jet pumps operate is generally limited to 7.6 m but may be more specifically limited by the depth where hard or cohesive sediment is found (Richardson and McNair 1981). Because scour holes in the Shinnecock Inlet channel have exceeded -12.2 m mlw, it should be safe to assume that the sand layer reaches at least this depth at the updrift fillet. However, diver observations in the scour hole indicated exposure of some clay sediments.[1] The maximum jet pump operating depth below natural bottom used for design should be 7.6 m. For practical purposes and because the jet pump will be operating at or near the mlw line, assume the 7.6-m depth is also a depth below mlw.

To minimize the potential for jetty undermining from encroachment of the jet pump crater on the structure, Richardson and McNair (1981) suggest a minimum distance between structure and edge of crater equal to $1.25 \times d$, where d is the design depth (7.6 m for Shinnecock). The rule of thumb for crater diameter given by Richardson and McNair (1981) is $3 \times d$. Therefore, at Shinnecock, the crater diameter should not exceed 22.8 m, and the edge of the crater should be no closer than 9.5 m to the base of the jetty. This distance equates to a practical jet pump operational limit of about 21 m from the jetty (Figure 23).

[1] Personal communication, Mr. Fred Anders, New York State Department of State, Albany, NY, 1996.

Identifying the pipe discharge location is important for identifying the pumping distance, overall length of pipe required, and whether booster pumps will be required. As discussed previously, pipeline discharge should be no closer to the inlet than the westward extent of the ebb shoal. According to Figure 14, this distance is approximately 2.4 km west of the west jetty. Including the distance required to cross the inlet (approximately 245 m), overall pipe length should be approximately 2,700 m.

If similar equipment as used at the Indian River Inlet plant is desired, then two booster pumps will be required (i.e., the primary booster at the operations building and one additional pump on the downdrift beach) (see Appendix F). Rule-of-thumb guidance provided by Turner (1984) states that the most appropriate location for placing the booster pump is at about 40 percent of the line length. In this case, the booster should be located 1,080 m downcoast of the primary booster pump. Alternatively, a single larger pump able to overcome the entire system head loss may be used to avoid having to operate two pumps. Details showing the calculations for head loss and pump power requirement are provided in Appendix F. The costs that follow assume using two pumps.

Operating time can be either daily (bypass plant is operated on a regular period each day), or intermittent (bypass plant is used only when bypassing is needed). Intermittent operation may include periods when the system is idle and times when the system operates continuously. For Shinnecock, a daily operating pattern is assumed for the period between September (after Labor Day) and May (before Memorial Day), not including weekends and holidays (approximately 251 days). Richardson and McNair (1981) suggest the following equation for calculating the effective operating time (EOT):

$$EOT = (NOD \times HD)[1.00 - (RR + PB + ALM + RMP)] \tag{7a}$$

where

EOT = effective operating time

NOD = number of operating days per year

HD = number of working hours in an operating day

RR = repair/replacement correction factor (÷100)

PB = pump blockage correction factor (÷100)

ALM = absence of littoral material correction factor (÷100)

RMP = relocation of mobile pump assemblies correction factor (÷100)

For Shinnecock, assume the following:

$$NOD = 251 \text{ total days} - 7 \text{ holidays} - 64 \text{ weekend days} = 180 \text{ days}$$

$$HD = 8 \text{ hours}$$

$$RR = 0.12$$

$$PB = 0.15$$

$$ALM = 0.03$$

$$RMP = 0.10$$

Therefore, at Shinnecock Inlet:

$$EOT = (180 \times 8)[1.00 - (0.12 + 0.15 + 0.03 + 0.10)] = 864 \ hr/year \tag{7b}$$

The PB, ALM, and RMP correction factors to some extent account for weather delays. But because no equipment is in the water, delays due to weather should be minimal. In fact, at Indian River Inlet, storm conditions are the preferred operating condition because of the increased transport being fed to the fillet. If the plant did not operate during storms, large quantities of drift would not be captured and thus lost to bypassing. Also, because no equipment is in the water, there should be no impact to normal navigation use (boating or dredging).

The target system capacity (the design operating rate in cubic meters per hour or cubic yards per hour) can thus be calculated as 116 m^3/hr (152 yd^3/hr) (100,000 m^3/year $\div$ 864 hr/year). However, for flexibility to bypass larger quantities if needed, the design capacity should be 150 m^3/hr (196 yd^3/hr), which equates to 129,600 m^3/year (170,000 yd^3/year). Additional capacity can be attained by increasing the number of hours worked in a day, increasing the number of days worked in a week, or expanding the operating time beyond the Labor Day to Memorial Day period.

A cost analysis for this alternative includes the cost to purchase each element as well as labor to operate the plant for a year. The equipment purchase cost (analogous to a mob/demob cost for floating plant) is amortized over a project life of 30 years using an interest rate of 7-3/8 percent. The estimate is based on similar costs of construction and operation for the Indian River Inlet bypass plant from HQUSACE (1991) and Delaware Department of Natural Resources and Environmental Control (Appendix G). At the time of this investigation (1997), construction costs (Table 15) were determined by comparing cost indices from 1983 and 1989 to 1997 as listed in Engineer Manual 1110-2-1304 (HQUSACE 1996).

Table 15
Construction Costs for Alternative 2: Semifixed Plant[1]

Item	Shinnecock Inlet Costs (1997 $)
Mob & Demob	35.8K
Operations Building	
1,500 SF @ $72/SF	108.0K
Utilities	11.1K
Security Measures	7.2K
Mechanical Equipment	
Water supply pump & engine	44.0K
Primary booster pump & engine	186.0K
Secondary booster pump & engine	186.0K
Booster pump pump house (1-200 SF @ $72/SF)	14.4K
Jet pumps (2@$50K each)	100.0K
Flushing water pump & engine	15.2K
Air compressor	12.2K
Instrumentation & gauges	21.5K
Crawler crane (135 ton)	620.0K
Vehicle	21.7K
Electrical Equipment	57.7K
Eductor & Discharge Piping	
12-in. HDPE (8,800 LF @ $10.75/LF)	94.6K
12-in. HDPE (2 @ 800 LF @ $10.75/LF) (line plugging redundancy crossing inlet)	17.2K
Butt-fusion equipment rental, training, etc.	2.1K
Pipe delivery charges	8.1K
HDPE & steel fittings	16.2K
Valves	44.0K
Installation	750.0K
Access Road & Parking Area	68.2K
Miscellaneous	64.4K
Subtotal	2.51M
Contingencies @ 20 percent	502.0K
Subtotal	3.012M
E&D @ 15 percent	452.0K
S&A @ 10 percent	301.0K
TOTAL	3.765M

[1] Cost details in Appendix G based on Indian River Inlet costs (which are also given in Appendix G) and other sources as noted.

Annual operating costs at Shinnecock Inlet are based on actual annual costs at Indian River Inlet from 1990 through 1995, which are detailed in Appendix G. The Indian River Inlet plant requires a three-person staff, but due to the added booster pump required at Shinnecock, a five-person crew is assumed. This crew size could be reduced by automating operation. The rationale and assumptions detailed in Appendix H are summarized in Table 16.

Table 16
Annual Operating Costs for Alternative 2: Semifixed Plant

Item	Shinnecock Inlet Costs (1997 $)
Operating crew	350.0K
Utilities	1.0K
Plant fuel	12.0K
Vehicles (incl fuel)	1.2K
Maintenance	54.0K
Total annual cost	418.2K

Finally, capital replacement and overhaul of various components of the bypass system must be factored into the costs. The Indian River Inlet replacement and overhaul costs and Shinnecock details are provided in Appendix I and summarized in Table 17.[1]

The overall annual cost of the bypass plant (including first cost, annual operating cost, and capital replacement cost) is calculated as:

First Cost + Operating Cost + Capital Replacement Cost = Total Annual Cost

$$\$3{,}765{,}000(0.0837)_{A/P,\,7\%,30} + \$418{,}200 + \$30{,}400 = \$763{,}731 \approx \$764{,}000 \qquad (8)$$

It should be noted that Indian River Inlet costs of $1.462 M in 1989, when converted to 1997 dollars (HQUSACE 1996) (21.2 percent increase), become $1.77 M. Discounting approximately $1.2 M in contingencies, engineering and design, and supervision and administration (which are not included in the Indian River Inlet figure) leaves approximately $2.6 M for initial construction of the Shinnecock semifixed alternative. The remaining difference of $830 K between the two sites can be largely attributed to a longer pumping distance (more pipe, two booster pumps, and increased labor).

[1] Personal communication, Mr. Robert Henry, Delaware Department of Natural Resources and Environmental Control, Dover, DE, 1996.

Table 17 Capital Replacement/Overhaul Costs for Alternative 2: Semifixed Plant			
Item	Interval (years)	Present Cost[1] ($)	Annual Cost[2] ($)
Crawler crane (incl components)	6 to 10	27.4K	2.3K
Diesel engines for booster pumps (two pumps)	20	9.5K	0.8K
Booster pumps components (two pumps)	2-4	108.0K	9.0K
Diesel engine for supply pump	16	5.0K	0.4K
Supply pump components	4-10	6.1K	0.5K
Air compressor & components	10-15	1.2K	0.1K
Gauges	5-10	4.9K	0.4K
Flow instrumentation	10	8.1K	0.7K
Crane-mounted density meter	15	0.7K	0.06K
¾-ton 4WD Diesel pickup	6	29.5K	2.5K
Slurry gate valve	6	1.8K	0.2K
All other gate valves	10	2.6K	0.2K
Jet pump & components	2-8	58.5K	4.9K
Pipeline & components	4-12	98.5K	8.2K
Pump house	20	1.4K	0.1K
Annual cost of capital replacement			30.4K

[1] Present cost (time value of money) of the periodic replacement/overhaul costs.
[2] Assumes an interest rate of 7-3/8 percent and 30-year return period.

Alternative 3: Land-based Mobile Plant (*Crawlcat/Crawldog*)

The third alternative considered is a land-based mobile dredge called *Crawlcat* or *Crawldog*. The *Crawlcat* is marketed by IHC Holland and consists of a floating pontoon with four legs on tracked wheels, which allows it to maneuver along the bottom in depths up to 10 m. Similar to a cutterhead dredge, the *Crawlcat* has a ladder with a dredgehead that pivots from side to side, and dredged material is discharged through a pipeline (Figure 24.) The *Crawlcat* has a narrow width because it was originally designed for use in canals with little or no wave activity. Though this design scheme has been used in an open coast environment, IHC Holland recommends against using the existing *Crawlcat* in such a manner.[1] Instead, a specialized

[1] Personal communication, Mr. Ruud Ouwerkerk, Dredge Technology Corporation, Wayne, NJ, 1996.

design, called *Crawldog* (to distinguish its design purpose from the *Crawlcat*) can be developed that would accommodate open coast conditions. This design would include a wider wheelbase for increased stability, two large tracks instead of four, and a lower (submerged) pump.[2] The *Crawldog* would offer the flexibility of moving within a borrow area (like a semifixed plant on a fillet) or being transported to a different location altogether. Because the *Crawldog* would be able to access much deeper water than a fixed or semifixed plant, it could increase the effective storage area of a fillet to areas well offshore. In some instances, it may even be able to access an ebb shoal directly.

Potential concerns for the use of the *Crawldog* include the exposure of the discharge line to breaking wave forces along the beachface and scour along (or burial of) the tracked wheels, which may cause the *Crawldog* to become stuck, especially if it must remain in one place for a long period (e.g. breakdown). Because of all the moving parts and long-term exposure of the tracked wheels to the marine environment, saltwater corrosion may pose a maintenance problem.

Typical *Crawlcat* designs (and probable *Crawldog* designs) include engine ratings of 350 to 550 kW (470 to 737 HP), pump capacities of 200 to 400 m^3/hr (260 to 520 yd^3/hr), and discharge pipe diameters ranging between 25 and 40 cm. Pumping distances depend on the application and size of *Crawlcat/Crawldog* used.

The following assumptions were used for cost analysis:

a. *Crawldog* will be purchased for $2.75 million.

b. Total *Crawldog* replacement will be needed in 15 years.

c. Unit cost of operation will be $5.50/$m^3$ ($4.21/$yd^3$) (which does not include additional booster pumps) (Bruun, in preparation).

d. Pipeline costs (initial and replacement) are the same as for Alternative 2.

e. Booster pumps and engine are the same as for Alternative 2.

f. The 3/4-ton, 4-WD diesel truck is the same as for Alternative 2.

Crawldog costs are summarized in Table 18.

Table 18
Cost Summary for Alternative 3: *Crawlcat/Crawldog*

Item	Present Value ($ 1997)	Annual Cost[1] ($)
Initial Cost		
Crawldog	2.75 M	230.0 K
Pipeline	452.2 K	37.8 K
Booster pumps & engines[2]	186.0 K	15.6 K
¾-ton 4-WD diesel truck	21.7 K	1.8 K
Storage facility (3,000 SF@ $75/SF)	225.0 K	18.8 K
Subtotal	3.635 M	534.0 K
With contingencies @ 40 percent	5.089 M	748.0 K
E&D (15 percent)	763.0 K	
S&A (10 percent)	509.0 K	
Initial cost	6.361 M	532.0 K
Annual Operating Costs[3]		550.0 K
Capital Replacement Costs		
Replacement *Crawldog*	2.75 M	230.0 K
Pipeline	98.5 K	8.2 K
Booster pumps & engines	58.7 K	4.9 K
¾-ton 4-WD diesel truck	29.5 K	2.5 K
Total Annual Cost		1.328 M

[1] Assume 7⅜ percent interest, 30-year return period.
[2] Assume primary booster is part of *Crawldog*; only one additional booster required.
[3] $\$5.50/m^3 \times 100{,}000\ m^3$ (includes labor).

Alternative 4: Submerged Mobile Plant (*Punaise*)

The fourth alternative examined is a submerged mobile system called *Punaise* (Brouwer, Visser, and van Berk (1991), Brouwer, van Berk, and Visser (1992), and Williams and Visser (1997)). The *Punaise* (Dutch for thumbtack) is a water-tight, submerged dredge pump system that can be lowered to the seafloor for sand/silt removal. It is controlled from a shore station that is connected by an umbilical line that contains the control connections, electrical power supply, and the slurry discharge line. Because the *Punaise* is located on the seafloor, it is able to operate in adverse wave conditions that prevent usual dredging/bypassing operations, and it does not affect normal navigation activities. In addition, it may be easily moved within the same general area, or it can be relocated to a different site altogether.

Dredging operations consist of positioning the *Punaise*, connecting it to the umbilical/discharge line, lowering it to the seafloor, and commencing dredging. Prior to positioning the *Punaise*, the shore station is established and the umbilical/discharge line is run from the shore station out into the water. Once the *Punaise* is positioned, it is connected to the umbilical/discharge line and its ballast tanks are filled to initiate sinking. As the *Punaise* sinks to the seafloor, the umbilical/discharge line also becomes submerged. Fluidizers in the suction pipe allow the *Punaise* to continue settling into the sand as it reaches the bottom. When the suction pipe becomes buried into the sand to the appropriate depth, dredging begins and a crater is formed under the *Punaise*. As material is removed from the crater, the *Punaise* can continue to settle until either the maximum depth of dredging is reached or the suction pipe encounters hard bottom (clay, gravel, rock, etc.) (Figure 25). Depending on the site and desired operation, the *Punaise* can either remain in place and continue to remove sand as the crater is filled, or it can be floated to the surface and repositioned to a new location.

Currently, there exist only two *Punaises*, which are operated in Europe by PinPoint Dredging Company, Inc. (a subsidiary of J.G. Nelis Group), The Netherlands. Specifics of these two systems are detailed in Table 19. Pumping distance, which is a function of grain size and production, can be estimated from Figures 26 and 27.

Table 19
***Punaise* Systems (PinPoint Dredging Company)**

Characteristic	PN250	PN400
Width	7.8 m	8.5 m
Height Without suction pipe With suction pipe	 3.1 m 8.5 m	 6.0 m 8.7 m
Draft	7.5 m	6.5 m
Pump capacity	800 m^3/hr @ 6 bar (1,046 yd^3/hr @ 87 psi)	2,400 m^3/hr @ 8 bar (3,140 yd^3/hr @ 116 psi)
Working depth	30 m	4 to 40 m
Discharge pipe diameter	26.0 cm	40.0 cm
Weight/Mass	47 metric tons	95 metric tons

Two scenarios exist for utilizing the *Punaise* at Shinnecock. The *Punaise* could be deployed in the subaqueous, updrift fillet with the shore station located on the updrift side of the inlet. Material dredged by the *Punaise* from the fillet (along 6.1-m contour) would be pumped to the shore station location and then across the inlet to the desired point of discharge. This total pumping distance is approximately 3,400 m (Figure 28). Assuming the *Punaise* PN400 can

pump a distance of approximately 2,000 m (capacity of 116 m^3/hr and extrapolating a d_{50}=0.4 mm from Figure 27), then one booster pump will be required to pump the material the remaining distance of 1,400 m (Appendix F).

The second scenario involves deploying the *Punaise* onto the ebb shoal from the downdrift beach along the 6.1-m contour with the shore station located on the downdrift beach. This total pumping distance is approximately 2,600 m (Figure 28). Assuming the *Punaise* PN400 can pump a distance of 2,000 m, then one booster pump will be required to pump the remaining 600 m (Appendix F). (Pumping distances of the *Punaise* PN400 are preliminary and may be extended with modifications to the system. Specifics about pumping distances and system operation should be addressed to PinPoint Dredging Company, Inc. , on a case-specific basis.)

A *Punaise* demonstration was planned for Shinnecock Inlet in January 1997. Total costs were estimated at \$810,000 to pump 153,000 m^3 (200,000 yd^3) of sand. Of this amount, approximately \$250,000 was for mob/demob, with the remaining \$560,000 for operation. The total unit cost for the demonstration project was to be \$5.29/m^3 (\$4.05/yd^3) or \$3.66/m^3 (\$2.80/yd^3) for operations.[1] PinPoint Dredging Company, Inc.[2] estimated an operational unit cost of \$2.80/yd^3 for a sand of d_{50}=0.3 mm pumped a distance of approximately 2,000 m. Including a booster station pushes the cost up to \$3.30/yd^3. Using these costs with booster pump costs used in Alternative 2: Semi-fixed, bypassing costs (Tables 20 and 21) for both scenarios can be estimated. Only lease/rental of the *Punaise* alternative is presented because PinPoint Dredging Company, Inc., has no plans to market the *Punaise* system for sales.

The demonstration project at Shinnecock Inlet was canceled in early January 1997, because a clay layer was discovered at the borrow site that was too close to the seafloor surface. This limited thickness of sand would prevent an efficient use of the *Punaise* in its current design at this particular borrow site. PinPoint Dredging Company is investigating a design modification that would make the *Punaise* more suitable for shallower water depths and/or thinner layers of sand. For the *Punaise* to be considered for use at Shinnecock Inlet, a thorough geophysical survey should be conducted to conclusively identify any potential borrow site where clay either runs deeper or does not exist to ensure sufficient sand layer thickness.

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[1]Personal communication, Mr. Fred Anders, New York State Department of State, Albany, NY, 1996.

[2]Personal communication, Mr. Kris Visser, PinPoint Dredging Company, Inc., (subsidiary of J.G. Nelis Group), Haarlem, The Netherlands, 1996.

Table 20 Cost Summary for Alternative 4: *Punaise* (Fillet Scenario)[1]		
Item	**First or Unit Cost (1997 $)**	**Annual Cost (1997 $)**
Geophysical survey	100 K	100.0 K
Mob/Demob	250 K	250.0 K
Operations	$4.32/m^3 for 100,000 m^3 (3.30/yd^3 for 131,000 yd^3)	432.0 K
Booster pump costs (1 pump)	186 K--purchase 4 K/year--fuel 4.9 K/year--capital replacement	15.6 K 4.0 K 4.9 K
Subtotal		806.5 K
With contingencies @ 30 percent		1.048 M
E&D (15 percent)		157.0 K
S&A (10 percent)		105.0 K
TOTAL		1.310M

[1]Assumes lease/rental of *Punaise* including pipeline.

Table 21 Cost Summary for Alternative 4: *Punaise* (Downdrift Scenario)[1]		
Item	**First or Unit Cost (1996 $)**	**Annual Cost (1996 $)**
Geophysical survey	100 K	100.0 K
Mob/Demob	250 K	250.0 K
Operations	$4.32/m^3 for 100,000 m^3 (3.30/yd^3 for 131,000 yd^3)	432.0 K
Booster pump costs (1 pump)	186 K--purchase 4 K/year--fuel 4.9 K/year--capital replacement	15.6 K 4.0 K 4.9 K
Subtotal		806.5 K
With Contingencies @ 30 percent		1.048 M
E&D (15 percent)		157.0 K
S&A (10 percent)		105.0 K
	TOTAL	1.310M

[1]Assumes lease/rental of *Punaise* including pipeline.

Alternative 5: Mechanical Fillet Mining

This alternative involves using a mechanical scraper/dragline or bucket system to physically move material from the offshore and nearshore areas of the updrift fillet to a holding basin near the back beach. From this holding basin, the material would be fluidized and pumped downdrift similarly to the previously discussed alternatives involving hydraulic pumping. For practical purposes, this alternative is the same as the semifixed alternative discussed previously with a different sand removal technique attached. Once the material is scraped to the holding basin, a jetpump/submerged pump system is required to fluidize the sand and, using booster pumps, pump it to the discharge site.

Draglines or scraper systems have been used in beach mining operations on less exposed coasts in the past (Renfroe 1971, Dalrymple 1970, Gee 1965). They consist of a land-based facility containing the drive motors and controls for the cabled bucket scraper. These cables are attached to an anchored, floating barge offshore which serves as the seaward limit of operation of the scraper system. In some instances, this offshore barge can be moored to allow movement up/downcoast for increased mining capacity (Figure 29).

The lack of installation experience in coastal regimes in recent years limits the confidence in the ability and practicality of implementation for bypassing using a mechanical system. In addition, there are several factors that may restrict or prohibit this type of system:

a. Suspension of sand causing undesirable plumes and increasing losses from the borrow area (Dalrymple 1970).

b. Anchoring a barge system connected by cables in an open ocean environment.

c. Corrosion of moving parts (gears, cables, and cable drums, etc.).

d. Aesthetics.

e. Cost unknowns (initial, operational, and replacement).

Because of the lack of recent experience, costs can only be loosely estimated. Gee (1965) provides an equipment cost in 1963 dollars of $40,000 for a system similar to that shown in Figure 29 for Jupiter Island, Florida. The unit cost to place 500,000 yd^3 of material on the beach was $0.24/$yd^3$. Accounting for a 343-percent increase from FY68 to FY97 (HQUSACE (1996) only goes back to FY68) and a 16-percent locality increase from Florida to New York , these costs would be $159,000 ($0.95/yd^3) in 1997 dollars. Note these costs are only for the dragline/ scraper portion of the system. Total costs will include the cost of fluidizing and pumping the material from the holding basin to the discharge point. Because this system is similar to the semifixed alternative, construction, operation and replacement costs will be similar as presented in Tables 15 to 17, less the crawler crane associated costs. Additionally, pipeline costs may also be reduced because the fluidization system will be immediately adjacent to the holding basin;

however, the construction of the holding basin will be an added cost. As an approximation, total costs of the mechanical fillet mining system would be at least as much as the semifixed alternative system and possibly higher, due to the added cost of dragline/scraper equipment discussed above and construction of the holding basin. For simplicity (and because the negatives discussed previously are significant enough to eliminate this alternative regardless of cost), the costs used for analysis will simply duplicate those of the semifixed alternative.

Summary

Annual costs of each of the five alternatives are summarized in Table 22. The relative higher cost of the floating plant alternative is initially somewhat surprising. However, when one considers that these costs are for relatively small volumes of material using the more expensive hopper dredge, then the costs seem reasonable. Additionally, many beach nourishment projects obtain their sand from channel maintenance projects where the cost of dredging is borne by the navigation interests and the cost of placement by the entities desiring sand on the beach.

Table 22
Cost Summary for Shinnecock Inlet Bypass Alternatives

Alternatives	First Cost/Mob & Demob ($)	Annual Cost ($)[1]
1--Floating Plant[2] Nearshore Placement	500 to 750 K	712 K (3-year cycle) 866 K (2-year cycle) 1.039 M (1-year cycle)
Onshore Placement	500 to 750 K	996 K (3-year cycle) 1.165 M (2-year cycle) 1.354 M (1-year cycle)
2--Semi-fixed Plant[3]	3.765 M	764 K
3--*Crawldog*[3]	6.361 M	1.328 M
4--*Punaise*[2] Fillet Scenario Downdrift Scenario	 250 K 250 K	 1.310 M 1.310 M
5--Mechanical Scraper/ Dragline	3.765 M	764 K

[1]Includes amortized first/mob-demob costs.
[2]Assumes lease/rental.
[3]Assumes purchase.

This cost is usually the difference between traditional lowest cost disposal options and beach or nearshore placement.

Among the intangibles associated with the semifixed alternative are concerns about line plugging and possible water hammer. The possibility of plugging the discharge line is a real danger, and every attempt to prevent plugging, including operating at a lower solids production

rate, should be examined, especially considering the distance of pumping. Generally, a plugged line can only be corrected by cutting open the line to physically clean out the sediment that has stopped flow. Cutting and cleaning out a plugged pipe is not uncommon, but it is not a minor task. A complicating factor is identifying exactly where the pipe is plugged. Given the semi-fixed alternative discussed here, without pressure gauges along the 2,700 m of pipe, the closest a plug could be identified would be between two booster pumps (i.e., within the 640-m section of pipe between booster pumps). Trial and error efforts to locate the plug location within a 640-m section of pipe could be significant.[1]

A plug in the pipe where the pipe is inaccessible (i.e., under the inlet) would render the entire system inoperable and may require specialized equipment or installation of another pipe crossing the inlet. A plug in the pipe under the inlet is of particular concern because even if solids production is low enough to minimize plugging from a system shutdown, the inlet crossing is the low point in the entire system. If flow ceased for any reason, there would be a tendency for slurry to travel "downhill" to the low point, which may create a plug under the inlet. Mr. Green has suggested that to minimize the impact of a plug under the inlet, two or three pipelines should be installed during initial construction with valves at the up- and downdrift side. Therefore, in the event a plug occurs under the inlet, the system can still be functional by switching to another pipe. Because the relative cost of additional pipe versus installation costs is much less, installing redundant pipes under the inlet will save future pipe installation costs by providing a safety fallback in the event of plugging, and will also extend the life of the pipe by allowing alternating between the pipes.

Water hammer is a phenomenon that could result from a near-instantaneous pipe blockage resulting from a booster shutdown or blocking off of the suction intake. When the flow is suddenly stopped at one location, a pressure wave traveling at the speed of sound travels in the direction opposite of initial flow until it reaches the previous booster pump, elbow, valve, etc. This pressure can be so great that pipes or even a pump casing can burst. Proper system design and pressure release valves will reduce the possibility of this occurring.

The line plugging and water hammer issues should be considered in all of the cases involving hydraulic pumping--semi-fixed, *Crawldog*, *Punaise*, and mechanical fillet mining. Neither of these problems are insurmountable, and in fact sand slurries are often hydraulically pumped along comparable distances in the mineral processing and phosphate industries. Much could be learned by the operators and designers of a bypassing project by examining pumping and pipeline systems in these industries.[1]

The *Crawldog* is the second-most-expensive of the alternatives on an annual basis (the most expensive being the 1-year cycle floating plant alternative with onshore placement), primarily because of the requirement for new design and construction (i.e., different from the *Crawlcat*).

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[1]Personal communication, Mr. Johnny Green, Standard Gravel Company, Franklinton, LA, 1996.

There are also certain unknowns associated with the first application of a new technology (such as potential design flaws and construction imperfections) that may make it less attractive. Though more mobile than a semi-fixed plant, the relative mobility of the larger *Crawldog* (i.e., compared to a floating plant or *Punaise*) would probably limit multiple inlet use.

The *Punaise*, which has the lowest first cost (mobilization/demobilization), has the fourth-highest annual cost, very near the *Crawldog* alternative. The Dutch experience over the past 5 years indicates that it is not a new technology (although it has not been used in the United States). *Punaise*, somewhat like a floating plant, also offers the intriguing possibility of use at multiple inlets. Mobilization and demobilization costs could be spread over several sites while the *Punaise* performs bypassing as needed/desired on a site-by-site basis. Also similar to the annual cycles described for the floating plant, the *Punaise* could pump larger quantities of sand over a larger time interval (2 or 3 years as opposed to 1 year) and thus reduce the impact of engineering and design, and contingencies. Several questions remain to be answered regarding the *Punaise* including running the umbilical through the surf zone and the stability of the *Punaise* to remain on the bottom during high wave conditions. Even with a modified design, the *Punaise* will not capture transport volumes moving shoreward of its location (currently around 6 m), which may prove to limit bypassing effectiveness.

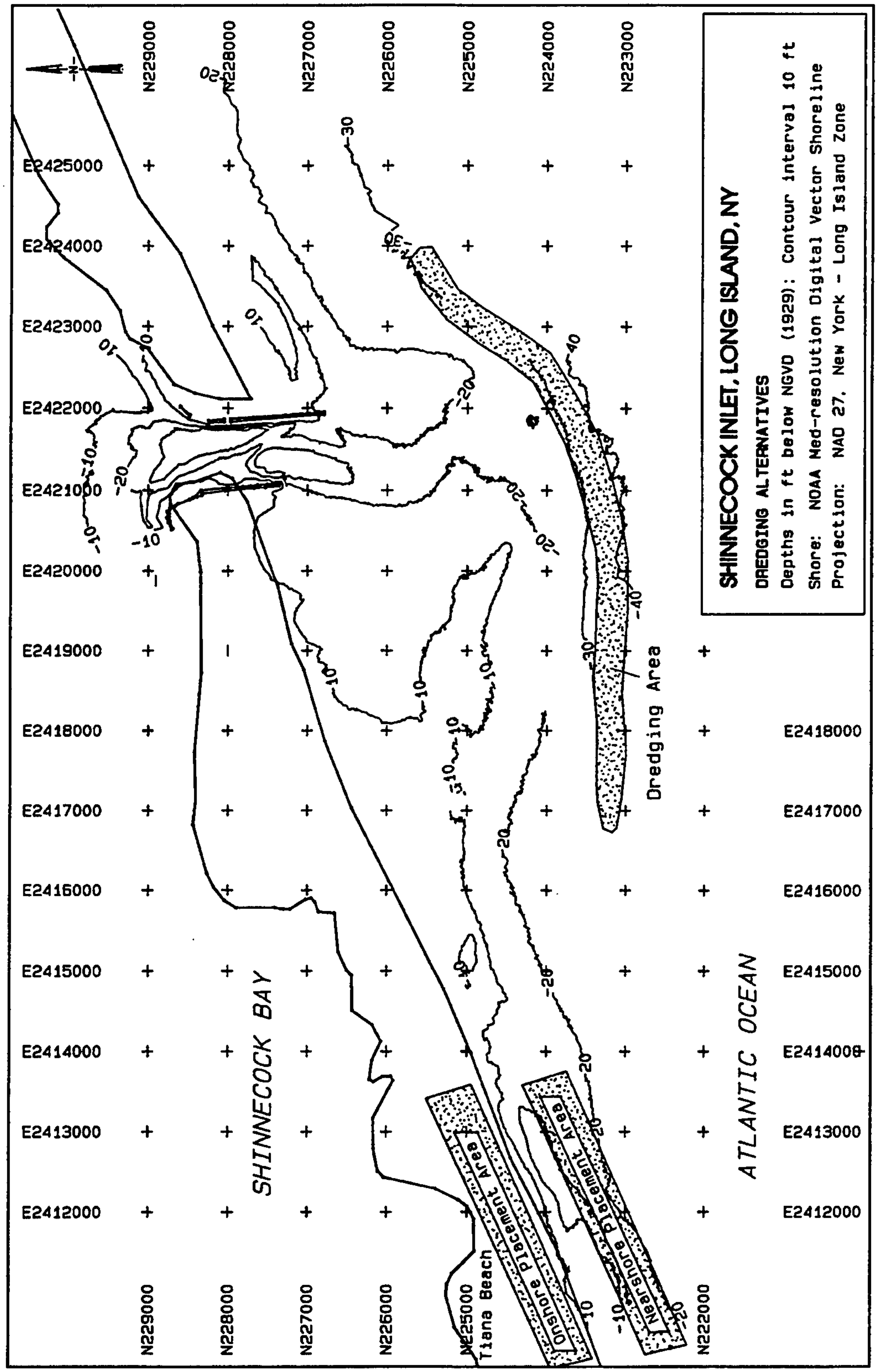

Figure 22. Potential dredging and placement locations–Alternative 1: Floating plant

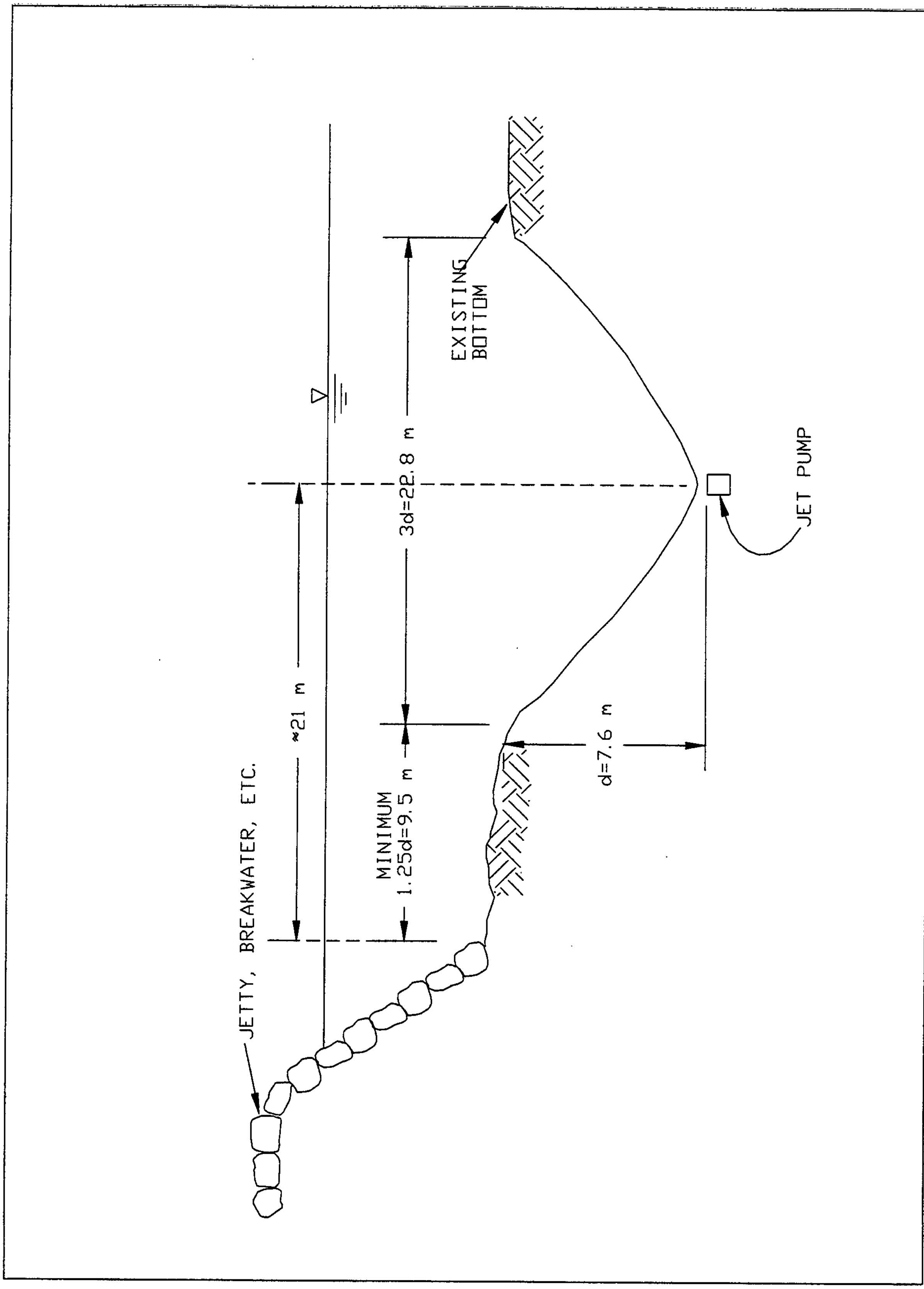

Figure 23. Location limit of jet pump operations with respect to jetty

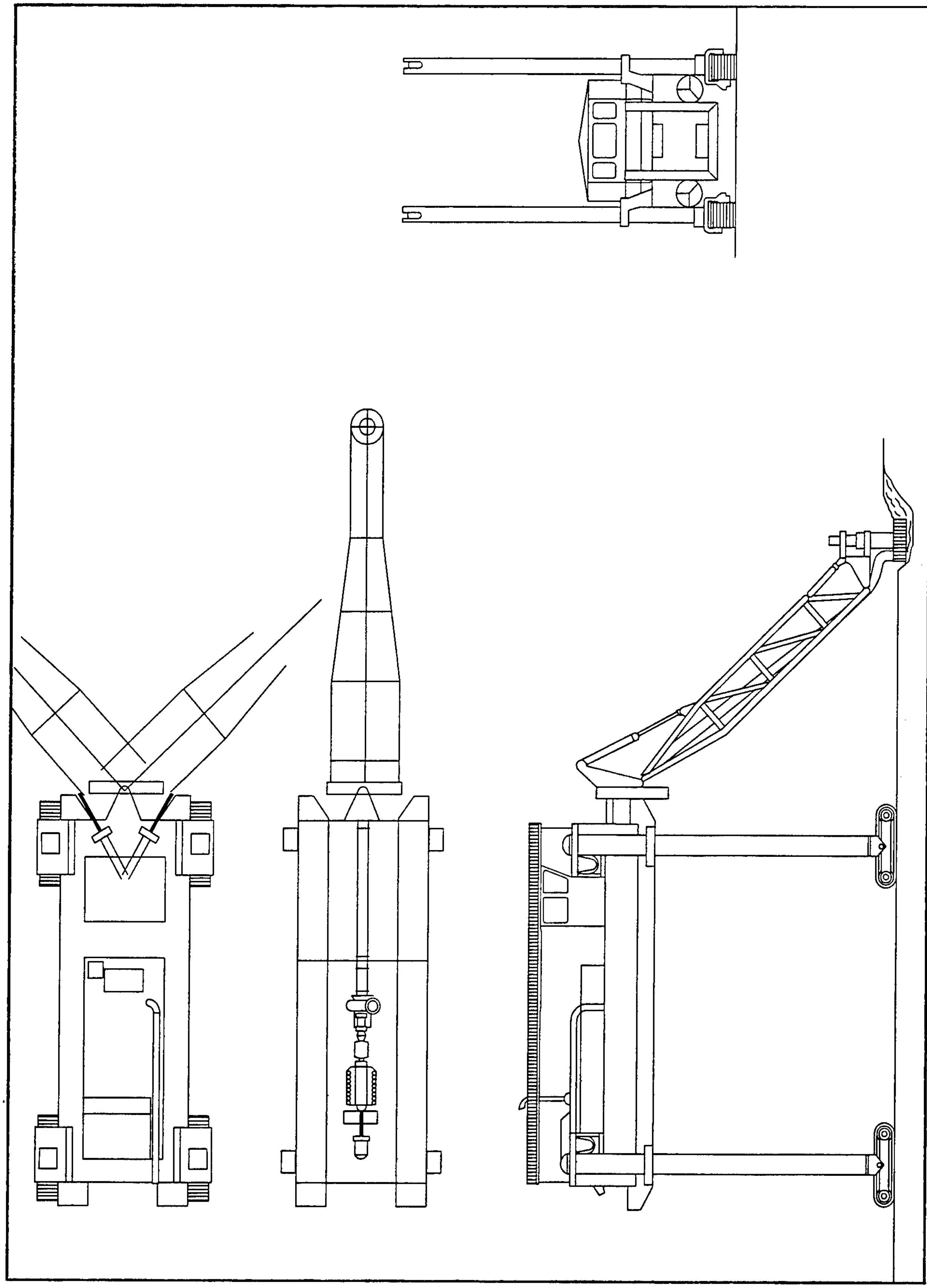

Figure 24. *Crawlcat* dredge (from IHC Group brochure)

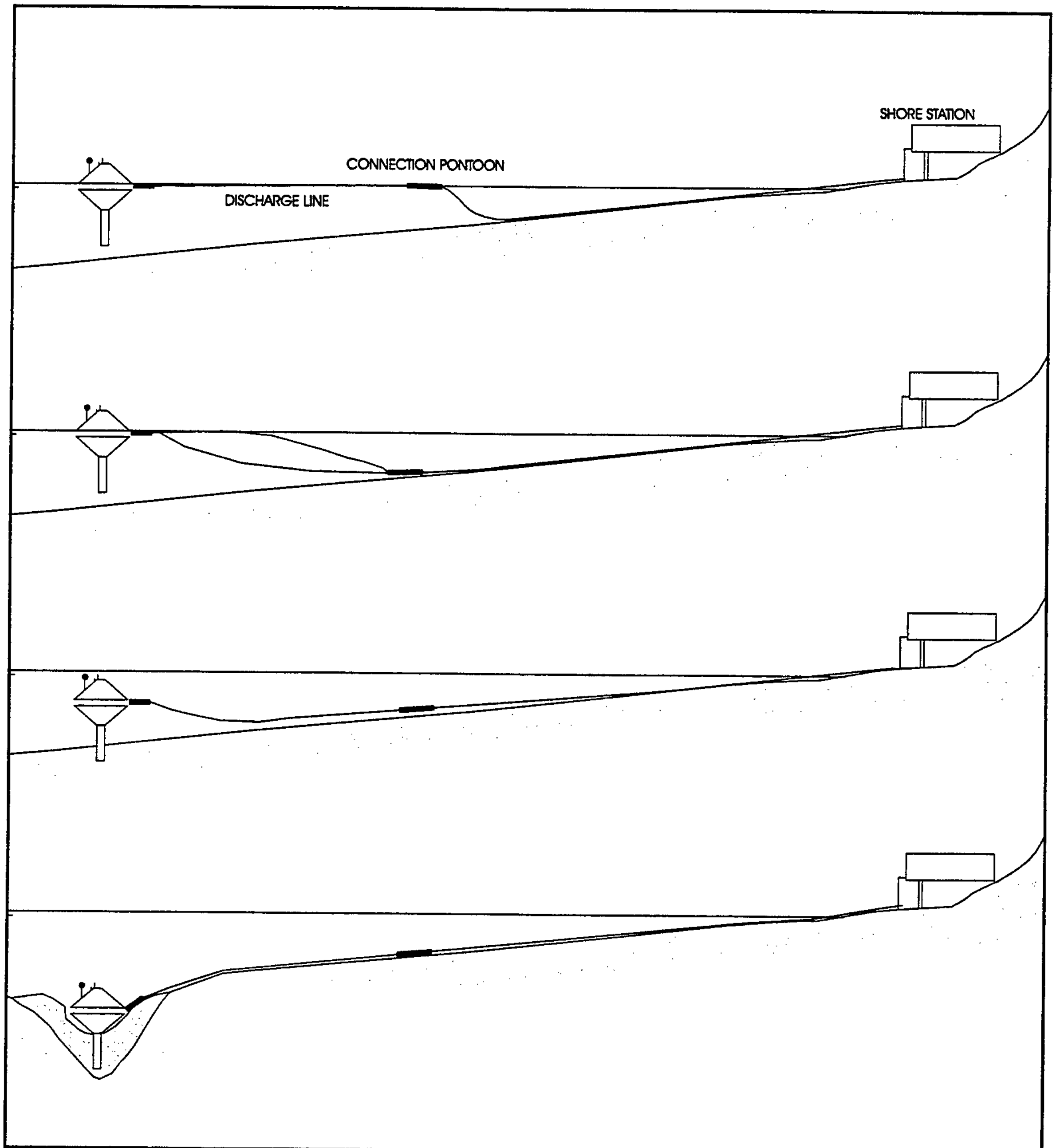

Figure 25. *Punaise* system operation scheme (Williams and Visser 1997)

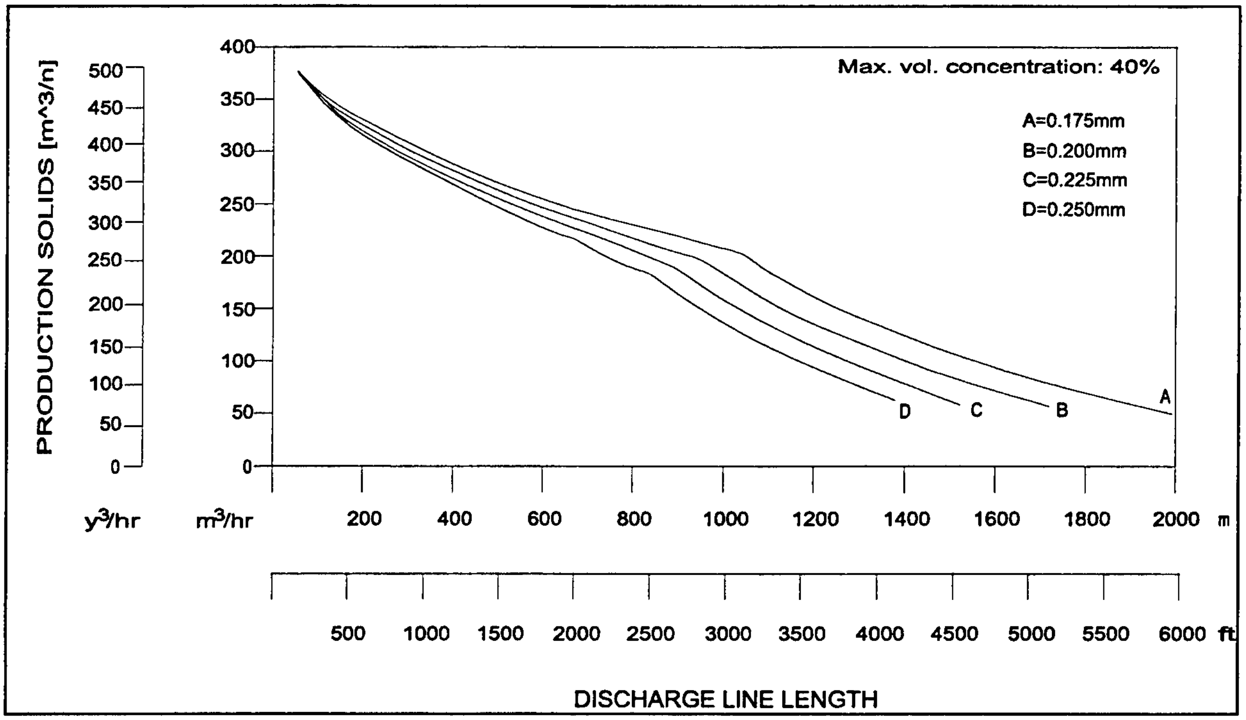

Figure 26. *Punaise* PN250 production rate versus discharge line length (Williams and Visser 1997)

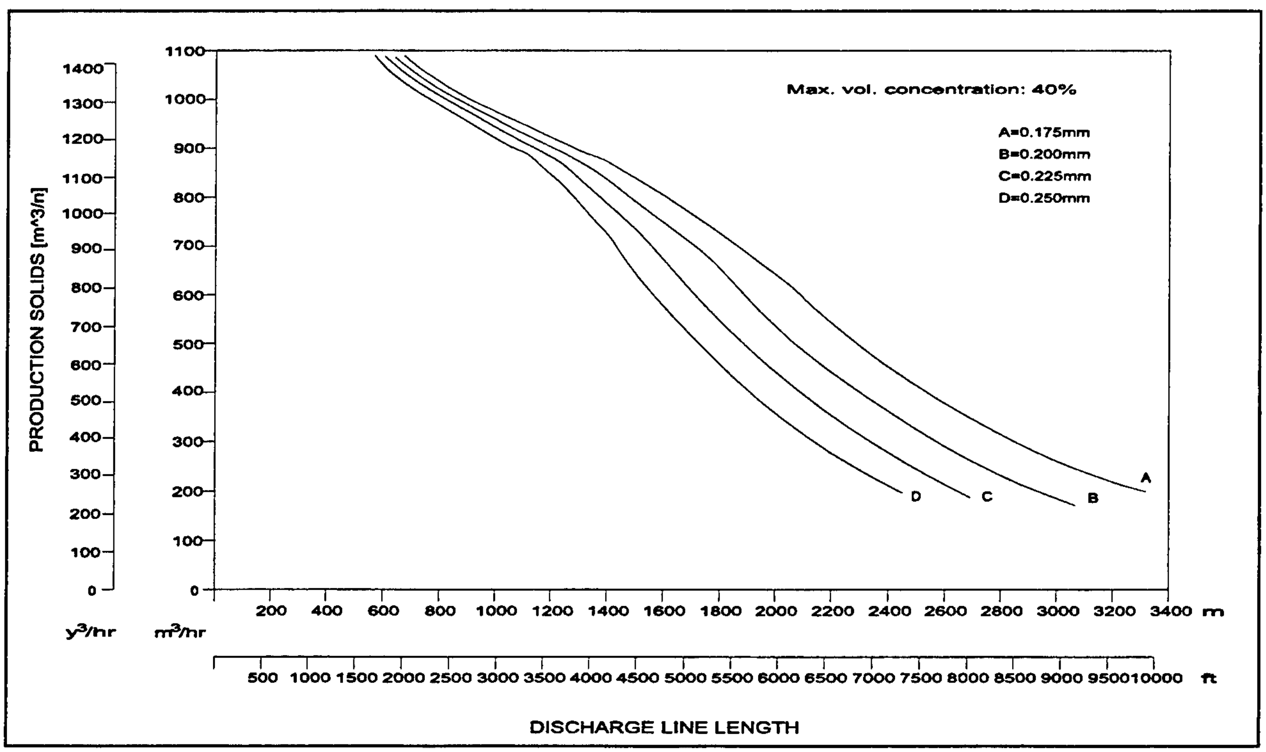

Figure 27. *Punaise* PN400 production rate versus discharge line length (Williams and Visser 1997)

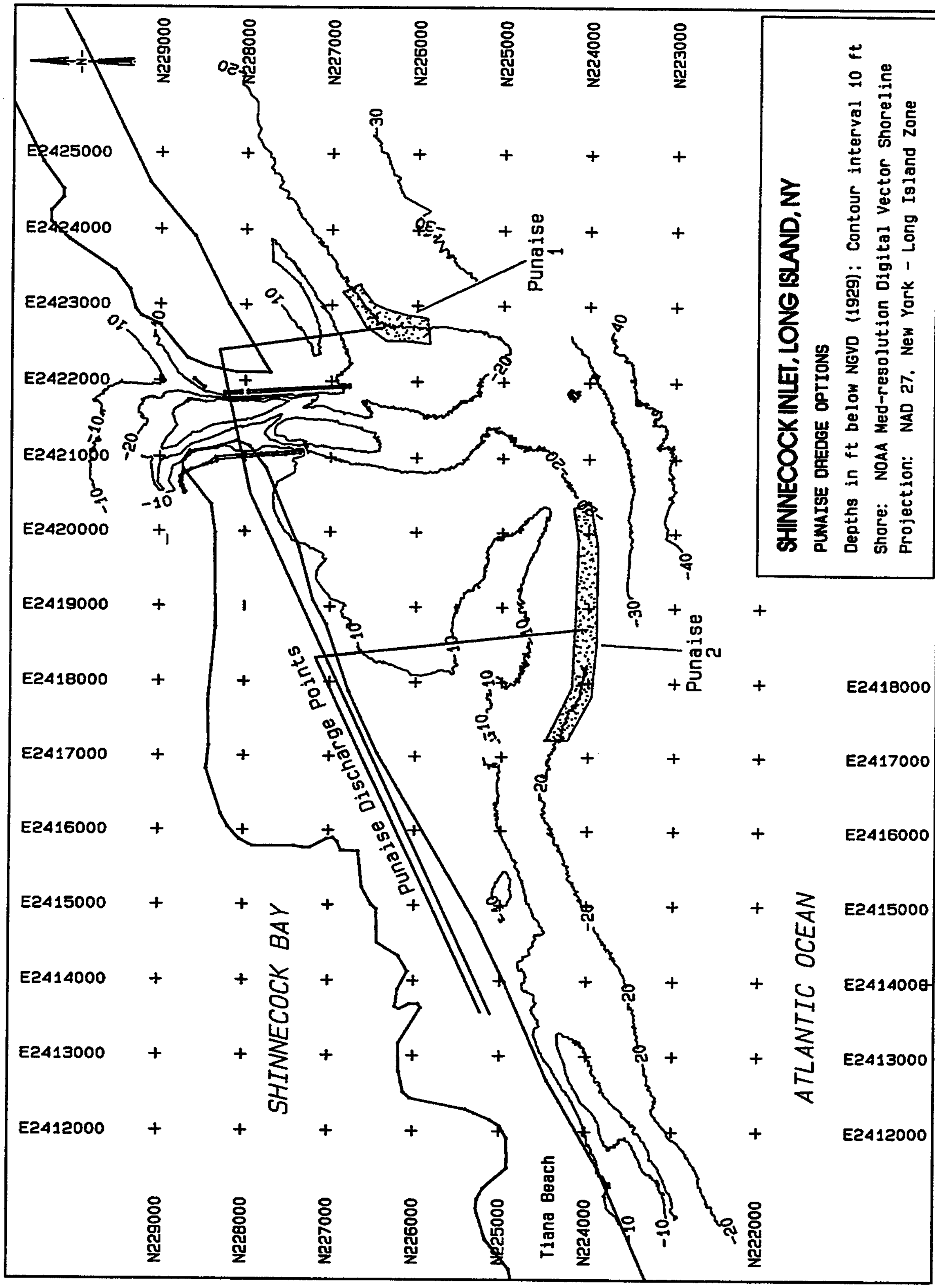

Figure 28. *Punaise* system implementation options

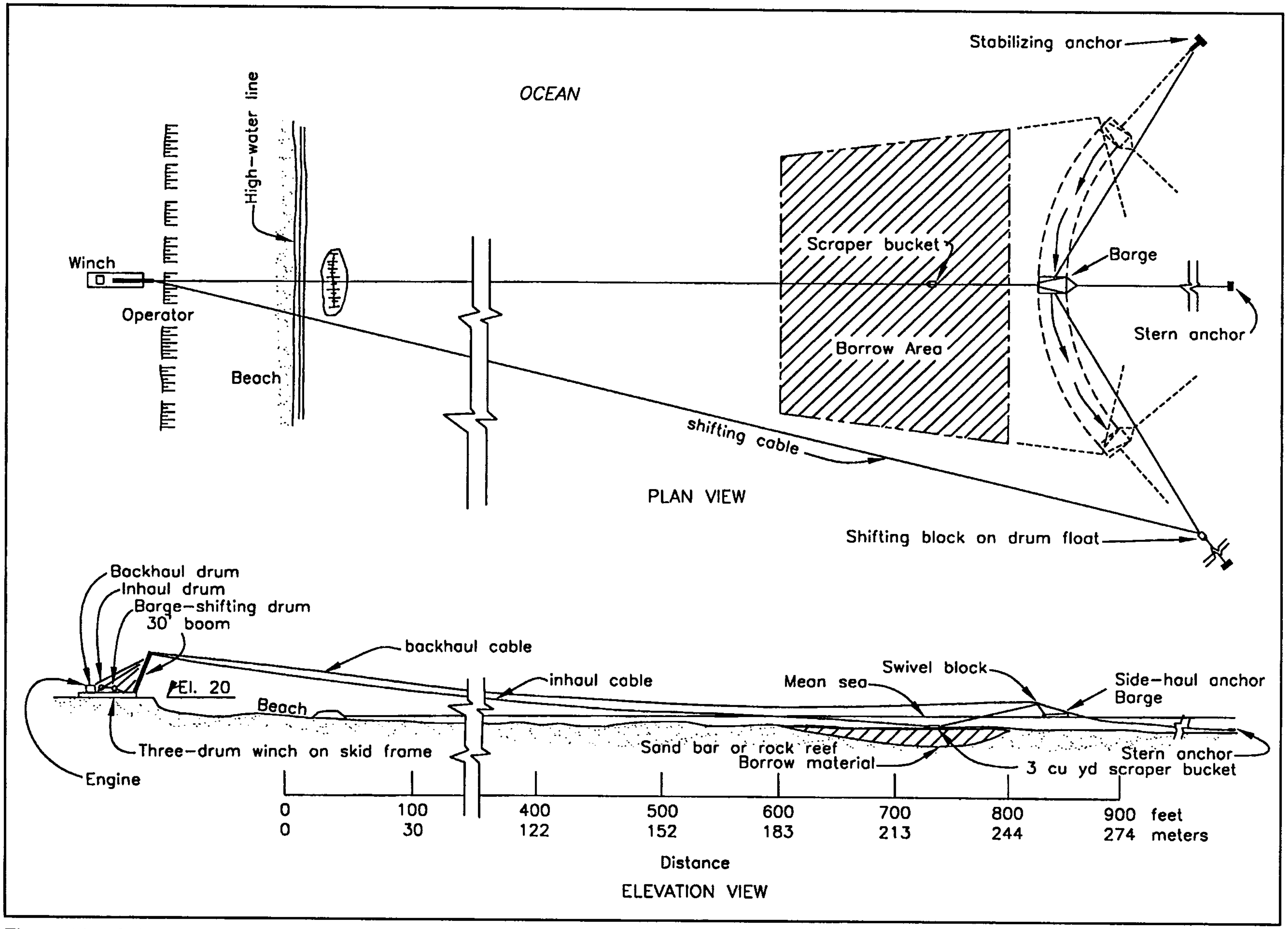

Figure 29. Schematic of a scraper/dragline system used in Florida (adapted from Gee (1965))

6 Other Bypassing Studies

A survey report by the USAE District, New York (1958) discusses improvements (dredging, jetty modifications, etc.) to Moriches and Shinnecock Inlets. This report also includes a preliminary plan for bypassing at both inlets. The proposed plan (specific for Moriches, but general for Shinnecock) is a fixed plant constructed with a pier extending seaward from the end of the east jetty to interrupt sand as it travels westward. The sand would then be pumped via a 10-in. discharge line under the inlet to a distance 450 m west of the west jetty. The plant would be capable of pumping 85 percent of the total 229,000 m^3 (300,000 yd^3) littoral drift but would only actually need to pump 96,000 m^3 (125,000 yd^3)--that amount not being bypassed naturally.

For development of a cost estimate for bypassing, USAE District, New York (1958) planned for 16-hr days 200 days of the year. Estimated first costs were $738,000 and annual costs were $155,700 (1957 dollars). Interestingly, converting these figures to 1997 dollars gives costs very similar to those generated for the semifixed analysis in the present study.

USAE District, New York (1958) also suggested that bypassing could be accomplished with a shallow-draft hopper dredge. Though no such dredge yet existed, NAN had tentative plans to construct a dredge with hopper capacity of approximately 230 m^3 (300 yd^3) and loaded draft of 1.8 m that could place material in the nearshore. Because this dredge was not yet built, no cost estimates were available.

PRC Engineering (1986) conducted a bypassing study at Shinnecock Inlet that evaluated seven alternatives, including three types of fixed dredge pump/jetpump systems, two semi-mobile systems, and two hydraulic dredging techniques. Unfortunately, PRC Engineering did not have the benefit of an updated, detailed coastal processes study for their analysis, so recommendations are based on a range of possible bypassing rates (between 45,900 m^3/year (60,000 yd^3/year) and 191,000 m^3/year (250,000 yd^3/year)). PRC Engineering also examined only the channel and updrift fillet as sources of sand and did not consider new technologies (which may not have existed in the 1980s).

PRC Engineering did recognize the need for placing the bypassed sand sufficiently downdrift to ensure restoration of regional transport. Their discharge location was approximately 1,600 m west of the inlet. But without accurate ebb shoal location data as exists for the current study, this

discharge location would now be approximately 800 m short of the western extent of the ebb shoal. This difference may well be attributed to an alongshore growth of the ebb shoal in the years between the two studies. This emphasizes the need for flexibility in bypassing.

PRC Engineering prepared a comparison table that graded each alternative on eight criteria (engineering, reliability, risk of damage, maintenance, operation, environmental, social, and aesthetic). Each of these criteria were weighted with engineering effectiveness (meeting operating schedules, productivity during rough seas, etc.) having the most weight, reliability, risk of damage, maintenance, and operation all having the next highest weight, environmental having the next highest weight, and social and aesthetic having the lowest weight. PRC Engineering's final recommendation showed that the floating dredge operating in the channel had the highest weighted score with the fixed dredge pump/jetpump alternatives being the next highest. When considering costs and bypassing rate, PRC Engineering recommended that for transport rates less than 46,000 m^3/year (60,000 yd^3/year), the hydraulic dredge was the best choice. For transport rates greater than 77,000 m^3/year (100,000 yd^3/year), the fixed jetpump system without crane was the best choice. PRC Engineering further recommended that for rates between 46,000 m^3/year (60,000 yd^3/year) and 77,000 m^3/year (100,000 yd^3/year), both methods should be more closely examined.

7 Conclusions

Downdrift erosion caused by the inlet and structures at Shinnecock Inlet can be addressed by mechanical bypassing. Several different bypass systems (five of which have been summarized here) are technically capable of restoring the natural littoral transport. Bypassing to restore natural longshore sediment transport has been successfully conducted at many inlets throughout the United States and the world, so the problem at Shinnecock is not insurmountable. One should consider, however, that restoration of longshore transport at Shinnecock Inlet may not solve all the beach erosion problems experienced to the west. Although interruption of the natural sediment transport is contributing to downdrift erosion, long-term barrier island retreat caused by sea level rise has also contributed to historical erosion (0.3 -1.2 m retreat per year--see Table 2). Mechanical bypassing can only mitigate those problems related to longshore sediment transport interruption, but it cannot solve the more widespread erosion/retreat problem caused by other mechanisms (i.e., sea level rise, lack of updrift sediment). Although artificial bypassing can restore longshore sediment transport to natural levels, this restored sediment supply may only solve the shore erosion problem where the material is placed and, with time, at adjacent reaches downdrift. Bypassing is most effective when beach nourishment is first used to bring the beach to its desired width; without the initial nourishment, bypassing can only maintain the existing condition.

Ideally, requirements for normal channel maintenance dredging should be reduced with any of the bypass systems presented. The level to which any one alternative can reduce channel shoaling cannot be determined from the information available for this study. Details on sediment transport pathways are required to determine the source of the sediment that shoals in the channel. Normal channel maintenance (if required) could continue as in the past with no impact to any of the bypass systems. Following a beach/nearshore placement of channel maintenance material, operation of the selected bypass system should be decided by the system operator with input from the downdrift benefactors and considering the season and existing conditions. One may argue that there should be no need to operate the fixed system for a time after channel maintenance operations if the material is placed downdrift. However, depending on the time frame of the maintenance dredging and the condition of the downdrift beaches, this may be viewed as an extra source of material providing an extra benefit. Alternatively, placement of maintenance material to supplement the bypassing system may allow a temporary shutdown of the bypass system to allow for maintenance or repairs (though the non-pumping season is also

ideal for equipment maintenance and repairs). Therefore, any channel maintenance conducted after implementing a fixed bypass option should be considered as providing an extra source of sand and be used accordingly in areas of greater need or as a supplement to the bypass system.

For the current study, to compare the bypassing alternatives discussed previously, an alternative selection decision matrix was developed. This matrix (Table 23) evaluates each of the five alternatives based on several criteria. Each alternative is assigned a numerical grade 1 to 5 (1 = most positive and 5 = least positive) for each of the criteria. These factors are then summed and normalized by multiplying by the unit cost of the alternative to give the final rating. Some assumptions inherent in this decision matrix are (a) all criteria are considered of equal weight, and (b) the grades (1-5) given to each criteria are subjective.

Criteria descriptions are as follows:

a. **Alternative experience**--amount of experience in the United States with this particular technology. Techniques like the *Crawldog* and *Punaise* are given higher grades because of their experimental nature and lack of experience. Floating plant is graded lower (more positive) because of the extensive experience with dredging in the United States.

b. **Navigation impact**--relative impact to ongoing navigation activities. Floating plants may have a higher grade, but because of the recommendation to use hopper dredges, which have a greater mobility, navigation impact is reduced. Semifixed, *Crawldog,* and mechanical fillet mining have minimal impacts to navigation because no equipment is in the channel. *Punaise* may have a slightly greater impact to navigation during repositioning, but because it resides on the bottom, impacts should be minor.

c. **Environmental impact**--relative impact to beach and underwater habitat for both sand removal and discharge/placement. Floating plant is graded lower because of the lack of permanent facilities on the beach, booster pump stations, and pipeline installations.

d. **Aesthetics**--the interference (or lack thereof) of visibility on/to the surrounding beach and ocean. Floating plant and *Punaise* are graded lower because of their transitory nature and lack of permanent facilities.

e. **Sand source flexibility**--flexibility of the alternative to remove sand from more than one location to minimize impacts of overuse. Floating plant and *Punaise* graded lower because of their mobility and potential to use the ebb shoal, flood shoal and/or updrift fillet. The *Punaise* may, however, be further limited by the thickness of the sand layer, as previously discussed. Semi-fixed, *Crawldog,* and mechanical fillet mining are all tied to the updrift fillet and thus graded higher.

Table 23
Alternative Selection Decision Matrix

Alternative	1st Cost ($1997)	Annual Cost ($1997)	Alternative Experience	Navigation Impact	Environmental Impact	Aesthetics	Sand Source Flexibility	Placement/Discharge Flexibility	Continuous=1; Periodic=5	Mechanical Reliability	Owner/Operator (Rent/lease=1; purchase/own=5)	Sum	Unit Cost (Annual Cost/100,000 m^3)	Final Rating
1–Floating Plant														
Nearshore (1 year)	1.039M	1.039M	2	2	2	2	2	2	5	1	1	19	10.39	197
Onshore (1 year)	1.354M	1.354M	1	2	2	2	2	2	5	1	1	18	13.54	244
Nearshore (2 year)	1.828M	866K	2	2	2	2	2	2	5	1	1	19	8.66	164
Onshore (2 year)	2.458M	1.165M	1	2	2	2	2	2	5	1	1	18	11.65	210
Nearshore (3 year)	2.367M	712K	2	2	2	2	2	2	5	1	1	19	7.12	135
Onshore (3year)	3.312M	996K	1	2	2	2	2	2	5	1	1	18	9.96	179
2–Semifixed	3.765M	764K	3	1	3	3	4	4	1	2	5	26	7.64	199
3–*Crawldog*	6.361M	1.328M	5	1	3	4	4	4	1	4	5	31	13.28	412
4–*Punaise*														
Fillet	250K	1.310M	4	2	3	2	3	4	3	3	1	25	13.10	328
Downdrift	250K	1.310M	4	2	3	2	3	4	3	3	1	25	13.10	328
5–Mechanical Fillet Mining	3.765M	764K	5	1	3	5	4	4	1	4	5	32	7.64	244

[1] Unit cost for Alternative #1 are from Table 14.
Note: Grades range from 1 (most desirable) to 5 (least desirable).

f. **Placement/discharge flexibility**--flexibility of the alternative to place/pump material to the nearshore, onshore, or along a large area. Floating plant graded lower because of mobility and ability to place nearshore and onshore. Other alternatives primarily limited to onshore placement with limited potential to adjust discharge outlet.

g. **Continuous/periodic**--ability of the alternative to mimic nature with essentially continuous bypassing. Floating plant graded higher because material would be delivered in large volumes over short periods. Semi-fixed, *Crawldog*, and mechanical fillet mining more closely resemble natural bypassing with a continuous/semicontinuous supply. *Punaise* graded in the middle because even though it pumps continuously/ semicontinuously, it may be remobilized to other locations, thus limiting ability to make unscheduled "start-ups."

h. **Mechanical reliability**--equipment performance based on knowledge of the operation of each alternative and the complexity of each alternative. Floating plant is the most established alternative and thus has the highest level of mechanical reliability. The semi-fixed alternative also has a high relative level of reliability because of the experience of use in the United States and relative simplicity of operation. The *Punaise* has a somewhat lower level of reliability because it is remotely operated, resides underwater, and there is relatively little experience in the United States with its use. *Crawldog* and mechanical fillet mining have the lowest reliability because of the mechanical complexity and lack of experience.

i. **Owner/operator**--ownership/operator of the alternative. Floating plant and *Punaise* graded low because of the ability to rent/lease equipment, thus preventing the need for full-time staff. Semifixed, *Crawldog,* and mechanical fillet mining all require permanent equipment and full-time employees.

If one looks strictly at annual costs, the semifixed plant may seem to be the most desirable alternative, except for the 3-year cycle (nearshore placement) floating plant. However, changing needs of the local sponsor and coastal processes and geomorphic changes over time may make a semifixed plant less desirable because it is fixed in one location and less able to allow for "mid-stream" corrections in operation. Physical changes that may warrant changes in the approach to bypassing would be less likely to be implemented because of the long-term commitment involved in construction and operation of this type of plant and the uncertainty of continued trapping of required quantities. Lease/rental arrangements for a floating plant or *Punaise* (lowest initial cost) system offer far more flexibility for changing conditions and demands. If physical, social, or political conditions at Shinnecock change in the future, a floating plant or *Punaise* would allow greater flexibility to change bypassing schemes and limit the loss associated with abandoning a capital investment. The floating plant (3-year cycle nearshore placement) provides the most flexibility for maximizing bypassing benefits and is the alternative that is most familiar in the United States.

The **Final Rating** from Table 23 shows that of all the alternatives, Alternative 1--floating plant (hopper dredge), on either a 2- or 3-year cycle, is the most desirable. If the periodic nature of operation of a floating plant is undesirable (i.e., if a somewhat continuous supply of sand is wanted), then Alternative 2–the semifixed alternative, is the most desirable (based on the decision matrix in Table 23). The least desirable alternative is Alternative 3--*Crawldog*. Note that all criteria were given equal weighting. Changing the subjective grades assigned to the criteria and/or weighting certain criteria more than others may change the selection results.

References

Allen, E. S. (1976). *A wind to shake the world.* Little Brown & Co., New York.

Alpine Ocean Seismic Surveys, Inc. (1997). "Final report: Vibracore sampling at Jones and Shinnecock Inlets, Long Island, New York," prepared for New York State Department of State, Division of Coastal Resources and Waterfront Revitalization.

Bodge, K. R., Creed, C. G., and Raichle, A. W. (1996). "Improving input wave data for use with shoreline change models," *Journal of Waterway, Port, Coastal and Ocean Engineering* 122(5), American Society of Civil Engineers, 259-63.

Brooks, R. M., and Brandon, W. M. (1995). "Hindcast wave information for the U.S. Atlantic Coast: Update 1976-1993 with hurricanes," Wave Information Study (WIS) Report 33, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Brouwer, J., Visser, K. G., and van Berk, H. (1991). "Silt removal with the Punaise, a flexible, self-burying dredging system." Proceedings, CEDA/PIANC Conference. Amsterdam, The Netherlands.

Brouwer, J., van Berk, H., and Visser, K.G. (1992). "The construction and nearshore use of the Punaise, a flexible submersible dredging system." *Proceedings, XIIIth World Dredging Congress*. Bombay, India.

Bruun, P. "New principles, methods and equipment for maintenance beaches and depths in channels and entrances," in preparation, submitted to ASCE *Journal of Waterway, Port, Coastal and Ocean Engineering*, Reston, VA.

Cialone, M. A., and Stauble, D. K. (1998). "Historical findings on ebb shoal mining," *Journal of Coastal Research* 14(2), 537-63.

Clausner, J. E., Gebert, J. A., Rambo, A. T., and Watson, K.D. (1991). "Sand bypassing at Indian River Inlet." *Proceedings of Coastal Sediments '91*. American Society of Civil Engineers, Reston, VA, 1177-91.

Clowes, E. S. (1939). *The Hurricane of 1938 on Eastern Long Island.* Hampton Press, Bridgehampton, NY.

Crowell, M., and Leatherman, S. P. (1985). "Quantitative shoreline and environmental change," *Final Report, Geomorphic Analysis, Fire Island Inlet to Montauk Point, Long Island, New York Reformulation Study.* S. P. Leatherman and J. R. Allen, eds., prepared for U.S. Army Engineer District, New York, 104-54.

Dalrymple, R. A. (1970). "An offshore beach nourishment scheme," *Proceedings of the 12th Coastal Engineering Conference.* American Society of Civil Engineers, Reston, VA, 955-66.

Dillon, W. P. (1970). "Submergence effects on Rhode Island barrier and lagoon and inference on migration of barriers," *Journal of Geology* 78, 94-106.

Fuller, M. L. (1914). "The geology of Long Island, New York," Professional Paper 82, Department of the Interior, United States Geological Survey, Government Printing Office, Washington, DC.

Gee, H. C. (1965). "Beach nourishment from offshore sources," *ASCE Journal of the Waterways and Harbors Division*, 91(WW3), New York.

Georgia Iron Works. (1982). *Slurry pipeline design manual.*

Headquarters, U.S. Army Corps of Engineers. (1991). "Sand bypassing system selection," Engineer Manual 1110-2-1616, Washington, DC.

__________. (1995). "Coastal geology," Engineer Manual 1110-2-1810, Washington, DC.

__________. (1996). "Civil Works Construction Cost Index System (CWCCIS)," Engineer Manual 110-2-1304, Washington, DC.

Leatherman, S. P., and Joneja, D. (1980). "Geomorphic analysis of south shore barriers, Long Island, New York: Phase I," National Park Service Cooperative Research Unit Report 47, The Environmental Institute, University of Massachusetts, Amherst, MA.

Lillycrop, W. J., Parson, L. E., and Irish, J. L. (1996). "Development and operation of the SHOALS airborne lidar hydrographic system," *Laser Remote Sensing of Natural Waters: From Theory to Practice*, SPIE - The International Society for Optical Engineering, vol. 2964, Bellingham, WA, 26-37.

McCormick, C. L. (1971). "Sediment distribution on the ebb and flood tidal deltas Shinnecock Inlet, Report 3," submitted to the Town of Southampton, NY.

McCormick, C. L., and Toscano, M. A. (1981). "Origin of the barrier island system of Long Island, N Y, *Northeast Geology* 3, 230-34.

Minsinger, E. E. (ed). (1988). The 1938 hurricane, an historical and pictorial summary. Blue Hill Observatory, East Milton, MA.

Moffatt & Nichol. (1996). "West of Shinnecock Inlet interim storm damage reduction project - draft report," Moffatt & Nichol Engineers, Baltimore, MD.

National Ocean Service. (1993). "Tide tables, high and low water predictions, east coast of North and South America, including Greenland," Washington, DC.

National Oceanic and Atmospheric Administration. (1987). "Shinnecock Inlet, Atlantic Ocean: Elevations of tidal datums referred to mean lower low water," Washington, DC.

Nersesian, G. K., and Bocamazo, L. M. (1992). "Design and construction of Shinnecock Inlet, New York." *Coastal Engineering Practice '92.* American Society of Civil Engineers, Reston, VA, 554-70.

Nummedal, D. 1983. "Barrier islands," *CRC Handbook of Coastal Processes and Erosion,* P. D. Komar (ed.), CRC Press, Inc., Boca Raton, FL, 77-121.

Osborne, E. G. (1970). "South shore inlets and place names," *Long Island Forum,* 88-90, 116-18.

Panageotou, W., Leatherman, S. D., and Dill, C. (1985). "Geophysical interpretation," *Geomorphic Analysis, Fire Island Inlet to Montauk Point, Long Island, New York, Reformulation Study.* S. P. Leatherman and J. R. Allen, eds., National Park Service, Boston, MA, for U.S. Army Engineer District, New York.

PRC Engineering. (1986.) "Beach Erosion Control and Hurricane Protection Fire Island to Montauk Point, Task 6 - Inlet By-Passing, Final Report," Report 02-1274-02, New York..

Providence Journal. (1938). "The great hurricane and tidal wave - Rhode Island - September 21, 1938," Providence Journal Company, Providence, RI.

Rampino, M. R., and Sanders, J. E. (1980). "Holocene transgression in south-central Long Island," *Journal of Sedimentary Petrology* 50 (4), 1063-80.

Renfroe, W. D. (1971). "A portable land-based dredge for no-man's-land areas." *Proceedings of World Dredging Conference (WODCON), 5th.* New Orleans, LA.

Research Planning Institute. (1983). "Sediment budget analysis, Fire Island Inlet to Montauk Point, Long Island, New York - reformulation study," report prepared for U.S. Army Engineer District, New York, by Research Planning Institute, Inc., Columbia, SC.

Richardson, T. W., and McNair, E. C. (1981). "A guide to the planning and hydraulic design of jet pump remedial sand bypassing systems," Instruction Report HL-81-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Sanders, J. E., and Kumar, N. (1975). "Evidence of shoreface retreat and in-place "drowning" during Holocene submergence of barriers, shelf off Fire Island, New York," *Geological Society of America Bulletin* 86, 65-76.

Schwartz, M. L. (ed.). (1973). *Barrier islands.* Dowden, Hutchinson & Ross, Stroudsburg, PA.

Shalowitz, A. L. (1964). *Shore and sea boundaries, with special reference to the interpretation and use of coast and geodetic survey data,* Vol. 2, Publication 10-1, U.S. Department of Commerce, Coast and Geodetic Survey, U.S. Government Printing Office, Washington, DC.

Shore protection manual. (1984). 4th ed., 2 vol, U.S. Army Engineer Waterways Experiment Station, U.S. Government Printing Office, Washington, DC.

Taney, N. E. (1961a). "Geomorphology of the south shore of Long Island, New York," Technical Memorandum No. 126, Beach Erosion Board, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

__________. (1961b). "Littoral materials of the south shore of Long Island, New York," Technical Memorandum No. 129, Beach Erosion Board, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Turner, T. M. (1984). *Fundamentals of hydraulic dredging.* Cornell Maritime Press, Centreville, MD.

U.S. Army Engineer District, New York. (1958). "Moriches and Shinnecock Inlet Survey Report," (original report dated September 1957; revised 11 July 1958), New York, NY.

__________. (1988). "General Design Memorandum Shinnecock Inlet Project, Long Island, New York," New York, NY.

U.S. Army Engineer District, Philadelphia. (1984). "General Design Memorandum and Environmental Assessment: Atlantic Coast of Delaware," Philadelphia, PA.

U.S. Army Engineer Waterways Experiment Station. (1991). "Memorandum for record: Tidal current measurement and side scan sonar survey of Moriches and Shinnecock Inlets, NY," Vicksburg, MS.

__________. (1993). "Memorandum for record: September 1993 tidal current study and side-scan sonar survey of Shinnecock Inlet, NY," Vicksburg, MS.

__________. (1995). "Memorandum for record: 1994 tidal current study and side-scan sonar survey of Shinnecock and Moriches Inlets, NY," Vicksburg, MS.

Walther, M. P., and Douglas, B. D. (1993). "Ebb shoal borrow area recovery," *Journal of Coastal Research, Special Issue No. 18*, 211-23.

Walton, T. L., and Adams, W. D. (1976). "Capacity of inlet outer bars to store sand." *Proceedings, 15th Coastal Engineering Conference (ICCE).* American Society of Civil Engineers, Reston, VA, 1919-37.

Whitford, N. E. (1906). *History of the canal system of the State of New York together with brief histories of the canals of the United States and Canada.* Brandow Printing Co., Albany, NY.

Williams, G. L., and Visser, K. G. (1997). "The Punaise: A remotely operated submerged dredging system." *Proceedings, Western Dredging Association (WEDA) 18th Technical Conference.* Charleston, SC.

Williams, S. J., and Meisburger, E. P. (1987). "Sand sources for the transgressive barrier coast of Long Island, New York: Evidence for landward transport of shelf sediments." *Proceedings, Coastal Sediments '77.* American Society of Civil Engineers, Reston, VA, 1517-32.

Appendix A
Chronological List of Geological Events

Table A1
Chronological List of Geological and Engineering Events at Shinnecock Inlet, Long Island, New York

Event	Date	Detail	Source
Early citation	1755	Seven inlets reported to be open east of Fire Island. Shinnecock Inlet probably open before 1755, according to Osborne (1970).[1]	Leatherman and Joneja 1980
Storm	Aug 19, 1788	Reported to be a "most terrifying storm." Possibly a hurricane, appears to have caused opening in Moriches Bay.	Leatherman and Joneja 1980
Storm	Dec 23-24, 1811	"The greatest blizzard of all time" caused severe damage to barrier islands.	Leatherman and Joneja 1980
Early citation	1829	Shinnecock Inlet shown open near the east end of the bay according to Osborne (1970).	Leatherman and Joneja 1980
Early citations	1830's	Shinnecock Inlet shown open on 1938-1950 U.S. charts according to Osborne (1970).	Leatherman and Joneja 1980
Barrier morphology	1838	USC&GS chart T-58 shows mainland at Quogue connected to barrier by low marsh, suggesting no open waterway between Quontuck and Shinnecock Bays. Bay shoreline is smooth along Tiana Beach and the island is rather narrow compared to present configuration.	Leatherman and Joneja 1980
Inlet open	Pre-1854	USC&GS chart shows Shinnecock Inlet open south of Rampasture, about 2.4 miles[2] west of present location. Islands in this area now may represent former flood shoal. Closed by May 1889.	Leatherman and Joneja 1980
Hurricane	Sep 8, 1869	Hurricane resulting in "unusually severe damage" was "the most severe storm since 1811."	Leatherman and Joneja 1980
1884-1892	Shinnecock and Peconic Canal	Cut through narrow neck of mainland between north end of Shinnecock Bay and Great Peconic Bay (traces existed of an earlier canal cut by Mongotucksee-Long Knife, Chief of the Mohawks). Funds provided by NY State Legislature. Purpose: flushing Shinnecock Bay to prevent stagnation and improve water quality to renew fish, clam, and oyster industries. Completed dimensions: 4,000 ft long, 58 ft wide at water surface, 4½ ft deep at low water. Project also intended to include cutting an inlet through barrier island to the Atlantic Ocean.	Whitford 1906
1896	Tide gates	Automatic tide gates built at south end of Shinnecock and Peconic Canal to keep Shinnecock Bay water level high and prevent erosion of banks and growth and decay of vegetation .	Whitford 1906

(Sheet 1 of 5)

[1]References cited in this appendix can be found in the list of references following the main text.

[2]A table of factors for converting non-SI units of measurement to SI units can be found on page x.

Table A1 (Continued)

Event	Date	Detail	Source
Inlet closed	1893-1933	Osborne (1970) stated Shinnecock Inlet closed this period. 1889-1890 USC&GS charts provide evidence of different inlets into Shinnecock Bay, all of which closed by 1891. One of the former openings was opposite Shinnecock Neck. Another was slightly west of Ponquogue Point. Two others were east and west of Gull Island, opposite East Quogue. 1903 and 1904 USGS surveys (Sag Harbor Quadrangle) showed no inlets into either Moriches or Shinnecock Bays.	Leatherman and Joneja 1980
Hurricane	Oct 10, 1894	Landfall around Moriches, caused severe damage.	Leatherman and Joneja 1980
Inlet cut	1896	As part of Shinnecock and Peconic canal project, channel 30 ft wide, 6 ft deep cut through the barrier island dunes with the purpose of increased flushing of Shinnecock Bay to relieve stagnant conditions. Local inhabitants donated services. Dunes said to be 20-30 ft high. "...It proved a failure, the waves quickly forming the dunes again, so that few traces of the channel now remain."	Whitford 1906
Hurricane	Sep 16, 1903	Damage to south shore of Long Island.	Leatherman and Joneja 1980
Hurricane	Sep 14-15, 1904	Damage to south shore of Long Island.	Leatherman and Joneja 1980
Storm	Mar 4, 1931	Reportedly led to reopening of Moriches Inlet. By 1933, inlet 1,300 ft wide.	Leatherman and Joneja 1980
Hurricane	Sep 21, 1938	The Great New England Hurricane, one of the most devastating storms in New England history, caused massive washovers all along south shore. Clowes (1939) described four inlets opening to Shinnecock Bay: 1. Near Warner's Islands, 0.5 miles east of Ponquogue Point, 40.5 miles east of Fire Island lighthouse. Closed 1938? 2. Opposite Cormorant Point, 41.6 miles east of lighthouse. By 1939, over 700 ft wide. Still open. 3. Opposite Shinnecock Hills, 43.3 miles east of lighthouse. Closed 1938? 4. Opposite Shinnecock Indian Reservation, 44.2 miles east of lighthouse. Closed 1938? See Figure 5 of this report.	Allen 1976, Leatherman and Joneja 1980
Bulkhead construction	1939	Suffolk County constructed 1,470-ft bulkhead on west side of inlet: timber piles, riprap, gabions, and 20 short spur jetties. Purpose: retard westward inlet migration.	Nersesian and Bocamazo 1992, USAED, New York 1988
Inlet morphology	1941	Inlet widened to the east to about 1,000 ft, inner and outer bar formed, tortuous channel connected ocean to Shinnecock Bay. Controlling depth only 4 ft.	Nersesian and Bocamazo 1992
Hurricane	Sep 14, 1944	Ravaged barrier islands.	Leatherman and Joneja 1980

(Sheet 2 of 5)

Table A1 (Continued)

Event	Date	Detail	Source
Repair	1947	800-ft stone revetment on west side and 130-ft stone groin added to north end by N.Y. State, Suffolk County, and Town of Southampton.	USAED, New York 1988
Storm	Nov 25, 1950	Three breaks (washovers) occurred east of Quogue, opening into Shinnecock Bay. A new inlet formed at Westhampton beach.	Leatherman and Joneja 1980
Channel	1951	Suffolk County dredged 9- by 200-ft channel 2,000 ft long through "inner sandbar" (flood shoal?).	USAED, New York 1988
Jetties	1953-1954	NY State, Suffolk County, and Town of Southampton built stone jetties on both sides of inlet. E. Jetty: 1,360 ft; W. Jetty: 850 ft (total length now 950 ft). Width of inlet fixed at 800 ft.	USAED, New York 1988, Nersesian and Bocamazo 1992
Hurricane Carol	Aug 31, 1954	Carol devastated east jetty and bayside revetment. Land adjacent to east jetty flooded by storm surge. Revetment damage caused by ebb of storm surge from bay. West of inlet, large zone of overwash extended across barrier island.	Nersesian and Bocamazo 1992
Channel	1958	Bay channel 10 by 200 ft dredged from inlet to Intracoastal Waterway by local interests. Subsequent maintenance in 1963, 1973, and 1978.	Nersesian and Bocamazo 1992
Hurricane Donna	Sep 12, 1960	Donna caused numerous washovers. By 1960, the Shinnecock jetties had resulted in little erosion or accretion. Well-defined shoal in bay.	Leatherman and Joneja 1980
Federal project authorized	1960	Existing project at Shinnecock Inlet adopted by the River and Harbor Act of July 14, 1960 (H. Doc 126, 86th Cong., 1st sess.). "This provides for an entrance channel 10 feet deep and 200 feet wide, from that depth in the Atlantic Ocean to Shinnecock Bay, thence an inner channel, 6 feet deep and 100 feet wide to the Long Island Intracoastal Waterway, rehabilitation of existing jetties and revetments, seaward extension of the west jetty about 900 feet, and construction of a fixed by-passing facility to transfer sand from the east side of the inlet to the west side." Authorized for three project purposes: 1. Navigation. 2. Water quality. 3. Beach erosion protection.	Ann. Rept. of Chief of Engr. 1961
Sediment budget	1961	BEB study concluded 300,000 yd^3/year to west.	Taney 1961
Cost estimate	1961	$3,551,000 estimate to complete work.	Ann. Rept. of Chief of Engr. 1962
Ash Wednesday Storm	Mar 4-6, 1962	Responsible for over 50 breaks (washovers) between Fire Island Inlet and Southampton. Notable offset at Shinnecock Inlet: west side eroded, accretion along east side.	Leatherman and Joneja 1980
Westhampton groins	1964-1966	11 groins built by USAED, NY, along Westhampton Beach (≈7 miles west of inlet).	USAED, New York 1988

(Sheet 3 of 5)

Table A1 (Continued)			
Event	**Date**	**Detail**	**Source**
East Hampton groins	1965	2 groins built by USAED, NY, at East Hampton (east of inlet). In addition, 2 smaller groins built by New York State.	USAED, New York 1988
Westhampton groins	1969-1970	4 more groins built west of 11-groin field at Westhampton Beach by USAED, NY.	USAED, New York 1988
Sediment budget	1983	RPI study commissioned for Reformulation Plan: 367,000 yd^3/year enters control volume; 247,000 yd^3/year leaves; approx 100,000 yd^3/year deposited on ebb shoal. (Note: RPI study considered deficient and generally unsatisfactory by USAED, NY, reviewers. Computations and conclusions have therefore not been used for planning or design.)	Research Planning Institute 1983
Dredging	1984	*Currituck* removed 176,000 yd^3 emergency dredging from various locations in inlet to -14 ft mlw. Disposal west of inlet at -10 ft mlw.	Project notes, Construction Div., USAED New York, 11/13/95
Dredging	Oct 1-23, 1990	668,000 yd^3 dredged from deposition basin (across ebb shoal). Disposal: 1. 138,000 yd^3 west of west jetty. 2. 77,000 yd^3 to fill scour hole by west jetty (channel side). 3. 193,000 yd^3 stockpiled on east side of inlet to use as fill behind revetment. 4. 260,000 yd^3 at Ponquogue Beach. Sand placed in scour hole lost within 1 year.	Project notes, Construction Div., USAED New York, 11/13/95
Deposition basin	1990 to 1993	Basin anticipated to fill with ≈ 425,000 yd^3 in 18 months. Unexpected result: almost no infilling over time. From 1990-93, <200,000 yd^3 was found in area, but not in prescribed basin.	Ms. Lynn Bocamazo, USAED, New York, personal communication, 12/10/97
Jetty repair	May 1992 to 1993	Repair of jetties, including construction of revetment along northeast shore of inlet within Shinnecock Bay to prevent erosion.	Ann. Rept. of Chief of Engr. 1992
Dredging	Jan 29 - May 14, 1993	475,000 yd^3 dredged from deposition basin (ebb shoal). Disposal: 1. 371,000 yd^3 west of W. Jetty. 2. 104,000 yd^3 to fill scour hole.	Project notes, Construction Div., USAED New York, 11/13/95
Dune Road repair	March 1993	Stone placed parallel to road. Beach filled between road and stone row.	Mr. Bill Daley, NY State Dep. Environmental Conservation, personal communication 12/10/97
SHOALS survey	Jun 1994	Survey of Shinnecock Inlet and ocean shore between Moriches and Shinnecock Inlets using USACE SHOALS helicopter LIDAR survey system. Uncertainty with tidal datum corrections.	CERC data
			(Sheet 4 of 5)

Table A1 (Concluded)

Event	Date	Detail	Source
SHOALS survey	May-Jun 1996	Survey of Shinnecock Inlet and extending along ocean shore to Westhampton Beach using SHOALS system. Greater coverage of ebb shoal than 1994 survey.	CERC data
Dune Road repair	Oct 1996	Northeaster caused erosion. State of NY repaired Dune Road with 14-16,000 yd^3 of sand brought in by truck.	Mr. Bill Daley, NY State Dep. Environmental Conservation, per. comm 12/10/97
Cores	Nov 1996	Five cores (one 40-ft and four 20-ft cores) taken at -6-m depth offshore of updrift fillet. Some clay layers detected. Proposed *Punaise* dredging tests cancelled.	Alpine Ocean Surveys, Inc., NY State Dep. of State
Channel dredging	Feb-Mar 1997	250,000 yd^3 placed west of west jetty. Material from dredging eastern flood shoal channel.	Mr. Bill Daley, NY State Dep. Environmental Conservation, personal communication 12/10/97
			(Sheet 5 of 5)

Appendix B Cartographic Data Analysis and Coordinate Conversion

Software used for all coordinate conversions and plotting of page-size charts:

Terramodel (Plus III Software, Inc., Atlanta, GA) DOS Version 8.40 for DOS-based personal computers.

Map products presented in this report:

Horizontal coordinates: State Plane New York Long Island zone, NAD 27, units in feet.[1]
Vertical units: Depths in feet below National Geodetic Vertical Datum (NGVD) or mean lower low water (mllw) (as marked on individual sheets). The shoreline represents the mean high water (mhw) line as depicted on National Oceanic and Atmospheric Administration (NOAA) hydrographic charts.

[1]A table of factors for converting non-SI units of measurement to SI units can be found on page x.

Table B1
Data Sources and Conversions

Data Type	Source	Original Coordinate System and Units	Terramodel Conversion
Shoreline	NOAA medium-resolution digital vector shoreline - mhw or mhhw line as shown on NOAA hydrographic charts.	Latitude longitude NAD 83.	COORDCON conversion from LL83 to NY-LI.
1933 bathymetry	NOAA hydrographic survey data from National Geophysical Data Center (provided in digital form).	Latitude longitude NAD 27; depths in m below mlw.	COORDCON conversion from LL27 to NY-LI; FACTZ multiply depths by 3.281; FACTZ add -1.66 ft to convert to NGVD (see Appendix E).
1949 bathymetry	U.S. Army Corps of Engineers (USACE) hydrographic and profile data, digitized from paper charts by Moffatt & Nichol Engineers.	Unknown. Supplied to U.S. Army Engineer Waterways Experiment Station (WES) in digitized form: State Plane, NAD27; depths in ft below NGVD.	COORDCON conversion from NY-LI 83 to NY-LI 27.
June 21, 1994 bathymetry	SHOALS LIDAR bathymetry survey from CERC archives. Includes inlet area and ocean coast between Moriches and Shinnecock Inlets.	Latitude longitude WGS 84; depths in m below mllw.	COORDCON conversion from LL84 to NY-LI; FACTZ multiply depths by 3.281. Total number of points reduced from ~160,000 to ~40,000 using FORTRAN program on VAX computer.
August - September 1996	SHOALS LIDAR bathymetry survey from John Chance & Associates. Includes inlet area and ocean coast off Westhampton Beach.	Latitude longitude WGS 84; depths in m below NGVD.	COORDCON conversion from LL84 to NY-LI; FACTZ multiply depths by 3.281. Total number of points reduced from ~492,000 to ~123,000 using FORTRAN program on VAX computer.

Appendix C
Adjustment of 1933 Hydrographic Data to Modern Datum

Adjustment for Sea Level Rise:

Battery, NY, sea level trend (from National Oceanic and Atmospheric Administration (NOAA) Internet site): 2.72 mm/year = 0.107 in./year = 0.0089 ft/year

Time interval: 1996 - 1933 = 63 years

Adjustment: 0.0089 ft/year * 63 year = 0.56 ft

Note: Water now is 0.56 ft *higher* than in 1933, so we must *add* 0.56 ft to water depths (i.e., if in 1933 a point was -10.0 ft, it now would be -10.56 ft assuming not changes in seabed) - see Figure E1a.

Datum:

At Shinnecock: Mean low water (MLW) to National Geodetic Vertical Datum (NGVD) (1929 adj.): 1.10 ft

Note: NGVD is 1.10 ft *higher* than mlw, so we must *add* 1.10 ft to depths (Figure E1b).

Total correction:

0.56 + 1.10 = 1.66 ft (Figure E1c)

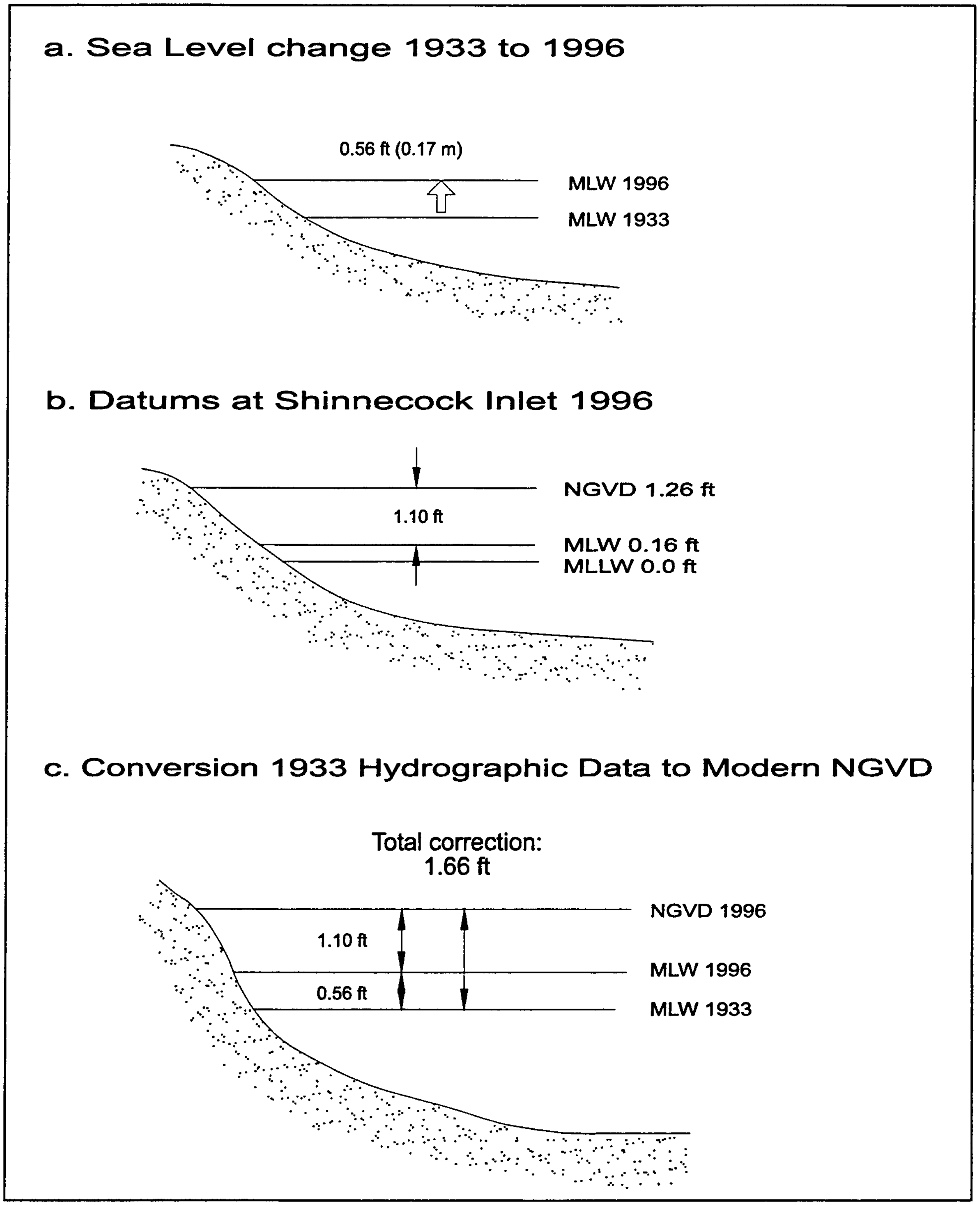

Figure C1. Conversion of 1933 hydrographic data to NGVD at Shinnecock Inlet

Appendix D
Simulation of 1949 Barrier Topography

Once Shinnecock Inlet was cut by the 1938 hurricane, the flood shoal grew rapidly. Volumetric computations (discussed in the main body of the report) show that between 1933 and 1949, about 2,200,000 m^3 of sand accumulated on the flood shoal. Some of this sand came from the barrier where the inlet was incised, while the rest came from interrupted littoral drift swept into the inlet on the flood tide (there is no significant amount of riverine sand brought into Shinnecock Bay). To determine sediment transport along the south shore, it was necessary to evaluate the relative proportion of barrier-derived sand versus littoral transport-derived sand.

Topographic information about the Long Island south shore in the Shinnecock area before the 1938 hurricane indicates that the barrier was continuous and consisted of dunes and marshy regions. Unfortunately, actual elevations are not shown on the early topographic sheets. The hurricane of 1938 is said to have "devastated the dunes," and post-storm aerial photographs show vast overwash areas, but these images provide little useful data regarding the prestorm morphology. Therefore, to determine a volume of sand eroded from the barrier, the pre-1938 topography was simulated.

To create a simulated barrier profile, the topography was measured across the contemporary barrier near Shinnecock Inlet along nine lines (Figure D1). These extended from the south shore waterline at an azimuth of 339 deg across the barrier to the north shore. The cross-barrier lines were spaced along the coast at the same positions as the beach and offshore profiles used for the Atlantic Coast of New York monitoring program. Topography was measured off detailed charts developed from 1995 aerial photographs (prepared for U.S. Army Engineer District, New York, by Erdman Anthony Engineers). The nine profiles were averaged in a computer spreadsheet program, and the average profile was used to simulate the pre-hurricane topography (Figure F2). The nine profiles included dunes with crest elevations ranging from 4 to 8 m and flat marshy zones.

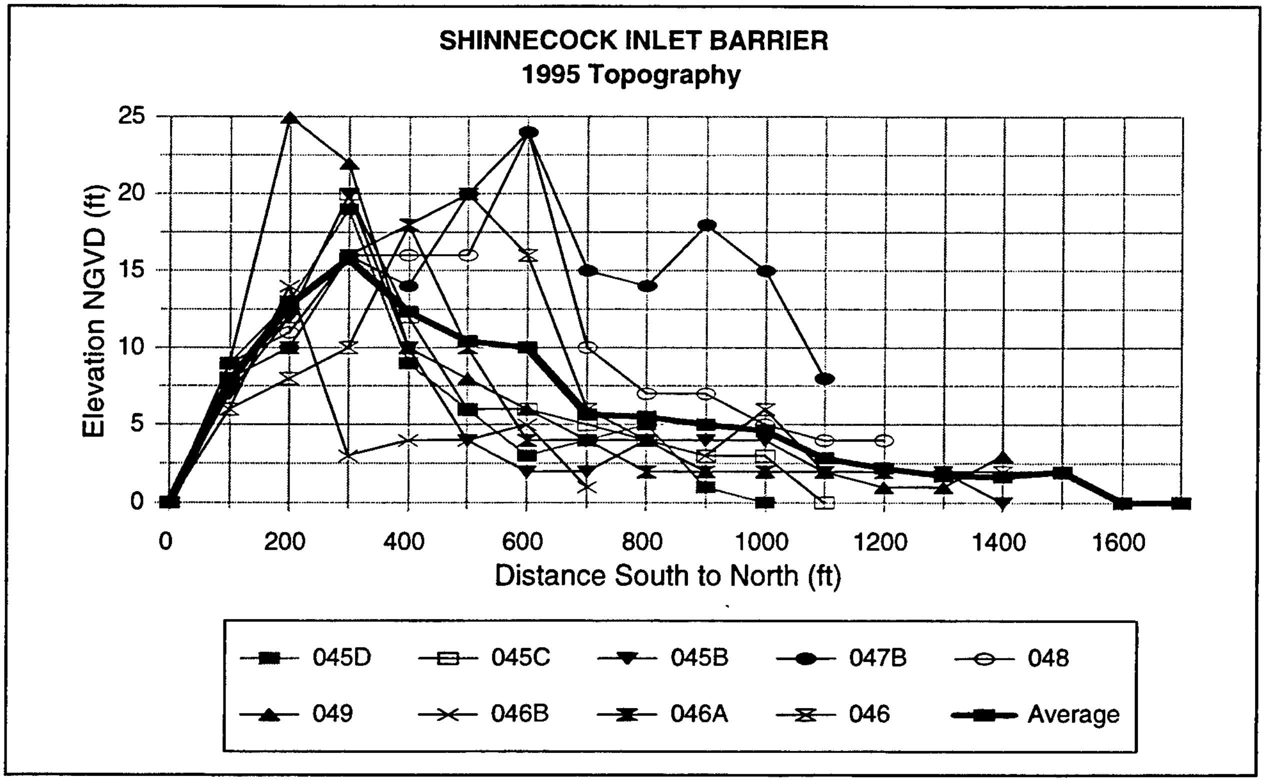

Figure D1. Profiles across the barrier near Shinnecock Inlet. Line numbers correspond to beach and offshore profiles regularly surveyed by the Atlantic Coast of New York monitoring program. Data measured from topographic sheets based on 1995 aerial photographs

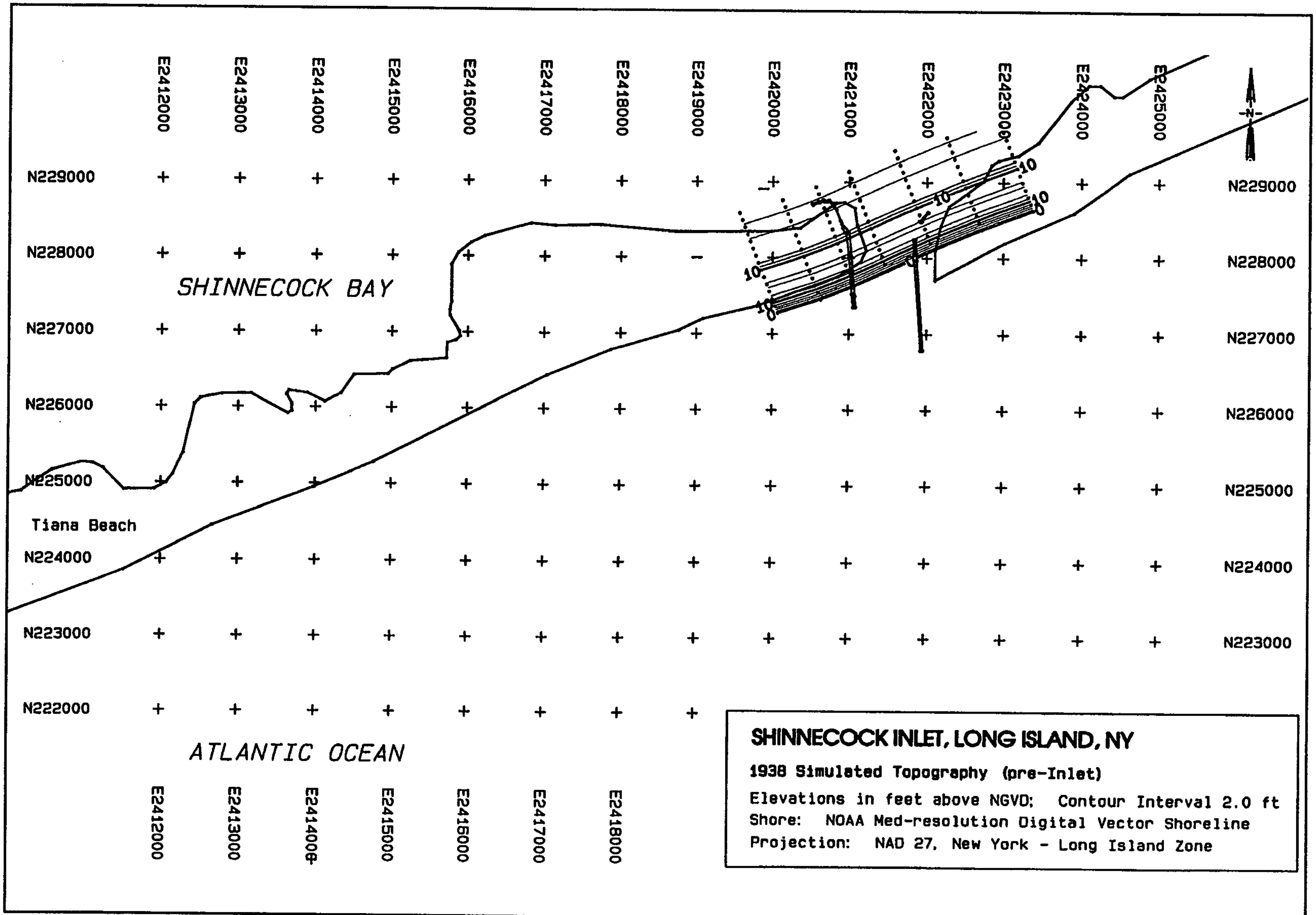

Figure D2. Simulated barrier topography, used in volumetric computations to determine amount of sand eroded from barrier when inlet was cut

Appendix E
Floating Plant Annual Cost Amoritization

This appendix shows the Present Worth and Capital Recovery Factors used in calculating the annual costs in Table 14 for the floating plant alternatives.

Table E1
Present Worth Factors for 2-Year Dredge Cycle for 30 Years (7-3/8 percent)

Year	PWF
2	0.867
4	0.752
6	0.653
8	0.566
10	0.491
12	0.426
14	0.370
16	0.321
18	0.279
20	0.242
22	0.261
24	0.182
26	0.158
28	0.137
Total PW	5.663

Table E2 Present Worth Factors for 3-Year Dredge Cycle for 30 Years (7-3/8 percent)	
Year	**PWF**
3	0.808
6	0.653
9	0.528
12	0.426
15	0.345
18	0.279
21	0.225
24	0.182
27	0.148
Total PW	3.594

The Capital Recovery Factor for interest rate of 7-3/8 percent for 30 years is 0.0837.

Table E3 Annual Costs			
Placement Option	**(PWF)*(CWF)**	**Project Cost ($)**	**Annual Cost ($)**
	2 Years		
Nearshore Placement	(5.663)*(0.0837)	1.828 M	866 K
Onshore Placement	(5.663)*(0.0837)	2.458 M	1.165 M
	3 Years		
Nearshore Placement	(3.594)*(0.0837)	2.367 M	712 K
Onshore Placement	(3.594)*(0.0837)	3.312 M	996 K

Appendix F
Booster Pump Worksheet

Great Lakes Dredge & Dock (GLDD) in Oakbrook, IL, was contacted for assistance in determining the number of booster pumps required for the semifixed plant alternative. The assumptions and their source or rationale are summarized as follows:

a. Assume sediment d_{50}=0.4 mm and d_{85}=0.8 mm from GLDD database for Shinnecock and Table 4.

b. Assume a pumping distance of 2,700 m (8,800 ft).

c. Assume a 12-in.-diam discharge line similar to what is used at Indian River Inlet.

d. Assume solids SG=2.65, slurry SG=1.15, in situ SG=1.90 (GLDD).

GLDD calculated a maximum critical velocity V_C = 10.1 ft/sec for d_{50} = 0.4 mm in a 12-in. pipe. To prevent settling, the pipeline velocity V should not be less than V_C. Therefore, for calculation purposes and conservativeness, let V = 11.5 fps.

Flow rate:

$$
\begin{aligned}
Q &= VA \\
Q &= V\left(\frac{\pi}{4}\right)d^2 \\
Q &= 11.5\left(\frac{\pi}{4}\right)(1)^2 \\
Q &= 9.03\ cfs = 4{,}053\ gpm = 1{,}204\ cyh
\end{aligned}
\tag{F1}
$$

Solids production:

$$Q_s = (1{,}204)0.15 = 180 \ cyh \tag{F2}$$

A standard method of calculating head loss in a pipe uses the Hazen-Williams Equation:

$$H = 0.2083 \left(\frac{100}{C}\right)^{1.85} \left(\frac{Q^{1.85}}{d^{4.8655}}\right) \tag{F3}$$

where:

H = friction head in feet of fresh water per 100 ft of pipe

d = inside diameter of pipe, inches

Q = flow rate, gallons per minute

C = constant describing pipe roughness based on d_{50} from Figure 11-2c, p. 92 in Turner (1984)

From Figure 11-2c (Turner 1984), using d_{50}=0.4 mm and slurry SG=1.15, C can be found equal to 110. Using the Hazen - Williams Equation, H can be found as:

$$H = 0.2083 \left(\frac{100}{C}\right)^{1.85} \left(\frac{Q^{1.85}}{d^{4.8655}}\right)$$

$$H = 0.2083 \left(\frac{100}{110}\right)^{1.85} \left(\frac{4{,}053^{1.85}}{12^{4.8655}}\right) \tag{F4}$$

$$H = 4.63 \ ft \ per \ 100 \ ft \ of \ pipe$$

$$H_{tot} = 407 \ ft$$

Georgia Iron Works (GIW) in their *Slurry Pipeline Design Manual* (September 1982)[1] provides a better method of determining head loss in feet of slurry as opposed to feet of clear water.

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[1]References sited in this appendix can be found in the list of references following the main text.

$$i_m = i_f + (S_{md} - 1)\left(\frac{U_u'}{V_m}\right)^{1.7} \tag{F5}$$

where:

i_m = head loss of slurry per length of discharge line (feet of slurry)

i_f = head loss of water per length of discharge line (feet), defined below

S_{md} = slurry specific gravity

U_u' = velocity factor dependent on grain size from Chart 4 in GIW *Slurry Pipeline Design Manual* (September 1982)

V_m = slurry velocity

$$i_f = \frac{fV^2}{2gd} \tag{F6}$$

where

f = friction factor as obtained from the Moody Diagram for pipe flow

V = pipe velocity

g = gravitational acceleration (32.2 ft/s^2)

d = pipe diameter

For the conditions at Shinnecock, *f* is determined to be approximately 0.012, *V*=11.5 fps and *d*=1 ft, so that i_f=0.0246. i_m is then found as follows:

$$i_m = 0.0246 + (1.15 - 1)\left(\frac{3.87}{11.5}\right)^{1.7}$$

$$i_m = 0.04185\ ft/ft \tag{F7}$$

$$H_{tot} = (0.04185)8{,}800 = 424\ ft$$

Assuming 65-percent pump efficiency for a slurry, the required horsepower to pump 4,053 gpm to overcome 424 ft of head can be calculated as follows:

$$HP = \frac{Q_{gpm}\ H}{3960(0.70)} = \frac{(4{,}053)(424)}{(3960)(0.65)} = 668\ HP \tag{F8}$$

For 8,800 ft of pipe and one pump operating at 668 HP, total head that must be overcome (or required head that pump(s) must provide) is 424 ft. If placing two pumps in series, head generated is additive (that is, two pumps generating 250 ft of head can together overcome 500 ft of total system head). Therefore, using the above equation for 250 ft of head (in place of the 424) gives required HP = 394. Therefore, two pumps generating 394 HP (similar to that used at Indian River Inlet) and able to overcome 250 ft can be used to accomplish the same pumping requirement. The 500-ft total head provided by the two pumps provides for 76 ft of head as a safety margin. However, certain labor and maintenance costs can be expected to be incurred when operating two pumps versus operating one pump. Cost comparisons between purchasing and operating two smaller pumps of the same size order as at Indian River Inlet versus one larger pump should be examined to identify the optimum operating scenario.

The *Punaise* alternatives will also require booster pump assistance for each scenario. The first *Punaise* scenario requires boosting for the remaining 1,400 m (4,600 ft) of discharge length. From the previous calculation using the GIW approach for head loss, 221 ft of loss will be expected, thus requiring one booster pump. For the second scenario, boosting will be required for the remaining 600 m (1,970 ft) of pipe discharge length causing 95 ft of head loss that must be overcome with a booster. All costs shown under the *Punaise* scenario include the cost of this additional booster pump.

Appendix G
Construction Cost Summaries for Alternative 2: Semifixed Plant

This appendix provides the background for the cost estimate for the semifixed alternative at Shinnecock Inlet given in Table 15. It is based on the Indian River Inlet costs as shown in pages G2-G6 from the General Design Memorandum and Environmental Assessment: Atlantic Coast of Delaware (U.S. Army Engineer District, Philadelphia 1984),[1] pages G7-G8 from "Sand Bypassing System Selection" (Engineer Manual 1110-2-1616 (Headquarters, U.S. Army Corps of Engineers (HQUSACE) 1991)) and other sources as noted. Many of the costs have been determined using the Indian River Inlet costs and converting to 1997 dollars using "Civil Works Construction Cost Index System (CWCCIS)," EM 1110-2-1304, dated January 1996 (HQUSACE 1996).

[1]References cited in this appendix are included in the list of references following the main text.

TABLE 8

TOTAL PROJECT COST ESTIMATE

Item No.	Description	Estimated Quantity	Unit	Unit Price	Estimated Amount
1	Mob & Demob Dredging		L.S.		$ 135,000
2	Dredging	80,000	C.Y.	3.96	$ 316,800
3	Mob & Demob Jet Pump System		L.S.		25,000
4	Operations Bldg.		L.S.		77,300
5	Mechanical Equipment		L.S.		$ 107,600
6	Electrical Equipment		L.S.		$ 39,500
7	Eductor & Discharge Piping		L.S.		$ 303,400
8	Miscellaneous		L.S.		$ 45,000
9	Access Road		L.S.		$ 11,500
			Subtotal		$1,061,100
		Contingencies @ 20%			$ 212,200
					$1,273,300
		E&D @ 15%			$ 191,000
		S&A @ 10%			$ 127,300
			Total		$1,591,600
			Total(rounded)		$1,592,000

TABLE 9

CONSTRUCTION COST BASIC BEACH FILL

Item No.	Description	Estimated Quantity	Unit	Unit Price	Estimated Amount
1	Mob & Demob			L.S.	$135,000
2	Dredging	80,000	C.Y.	3.96	316,800
			Subtotal		$451,800
		Contingencies @ 20%			$ 90,360
			Subtotal		$542,160
			E&D @ 15%		$ 81,320
			S&A @ 10%		$ 54,220
			Total		$677,700
			Total (rounded)		$678,000

TABLE 10
JET PUMP SYSTEM
DETAILED COST ESTIMATE

Item No.	Description	Estimated Quantity	Unit	Unit Price	Estimated Amount
1	Mob & Demob		LS		$ 25,000
2	Operations Bldg.		LS		
	Structure	1,290	SF	50.00	$ 64,500
	Utility Service		LS	—	$ 7,740
	Security Measures		LS		$ 5,000
			Subtotal		$ 77,240
3	Mechanical Equip				
	Water Supply Pump & Motor	1	Each		$ 22,300
	Booster Pump & Motor	1	Each		$ 44,150
	Flushing Water Pump & Motor	1	Each		$ 10,600
	Eductors	2	Each	3,510	$ 7,020
	Air Compressor	1	Each		$ 8,530
	Instrumentation & Gages		LS		$ 15,000
			Subtotal		$107,600
4	Electrical Equipment				
	Transformer 500KVA	1	Each		$ 12,100
	Transformer 480/240/120	1	Each		$ 970
	Motor Control Center		LS		$ 12,530
	Underground Cables		LS		$ 13,840
			Subtotal		$ 39,440
5	Eductor & Discharge Piping				
	8" ϕHDPE	1000	LF	7.25	$ 7,250
	10" ϕHDPE	5790	LF	11.24	65,080
	14" ϕHDPE	40	LF	19.00	760
	HDPE Fittings		LS		$ 1,610
	2-1/2" ϕSteel	10	LF	3.00	230
	2" ϕ Steel	10	LF	18.10	180
	20" Steel	40	LF	130.00	5,200
	Steel Fittings		LS		4,080
	2-1/2" ϕ Gate Value	1	EA		200
	2" ϕ Gate Value	1	EA		150
	10" ϕ Gate Value	17	EA	1300.00	22,100
	14" ϕ Gate Value	1	EA		2,400
	2" ϕ Check Value	1	EA		120
	10" ϕ Check Value	2	EA	1250.00	2,500
	14" ϕ Check Value	1	EA		2,200
	Air Relief Value	1	EA		1,000
	Installation		LS		188,300
			Subtotal		$303,360

TABLE 10 (Continued)

Item No.	Description	Estimated Quantity	Unit	Unit Price	Estimated Amount
6	Miscellaneous				
	Flotation Tanks	10	Ea	3,000	$ 30,000
	Buoys		LS		5,000
	Miscellaneous		LS		10,000
				Subtotal	$ 45,000
7	Access Road & Parking Area				
	Excavation & Preparation	140	CY	10.00	$ 1,400
	Wearing Course 2-1/2"	630	SY	6.70	4,220
	Base Course 4"	630	SY	4.90	3,090
	Subbase 6"	630	SY	4.40	2,770
				Subtotal	$11,480
				Subtotal	609,120
				Contingencies @ 20%	121,820
				Subtotal	730,940
				E&D @ 15%	109,640
				S&A @ 10%	73,090
				Total	$913,670
				Total (Rounded)	$914,000

TABLE 11
ANNUAL OPERATION AND MAINTENANCE COSTS
JET PUMP SYSTEM

A	Operating Crew (2 persons)	$75,000
B	Building Maintenance	3,600
C	Equipment Maintenance	26,000
D	Materials & Supplies	10,000
E	Vehicles	4,000
F	Utilities	47,900
G	Periodic Inspection	6,000
	Subtotal	$172,500
	Contingencies @ 10%	17,300
	Subtotal	$189,800
	E&D @ 7%	13,280
	S&A @ 5%	9,490
	Total	$212,570
	Total (rounded)	$ 213,000

TABLE 12
PROJECT COST COMPARISON

	Re-evaluation Report Oct 83 Price Level	GDM Oct 84 Price Level
Initial Beach Fill	$447,500	$542,200
Jet Pump System	598,750	730,900
Groin	287,500	--
Engineering & Design	200,060	191,000
Supervision & Administration	133,380	127,300
TOTAL PROJECT COST (Rounded)	$ 1,668,000	$1,592,000
Annual Operation and Maintenance (Rounded)	$214,000	$213,000

TABLE 13
COST ESTIMATE SURVEILLANCE PLAN

	First 5 Years	After first 5 years
Profiles	$18,400	$ 8,800
Aerial Photography	8,800	4,400
Data Compilation	2,000	1,500
Subtotal	$29,200	$14,700
Contingencies @ 10%	2,920	1,470
Subtotal	$32,120	$16,170
E&D @ 7%	2,250	1,130
S&A @ 5%	1,610	810
Annual Total cost (rounded)	$36,000	$18,000

APPENDIX C

MAJOR COMPONENTS AND PRICES FOR THE INDIAN RIVER INLET, DELAWARE, SAND BYPASS SYSTEM

C-1. General. This appendix summarizes first costs associated with various bypassing system components for the Indian River Inlet, Delaware, bypassing project, a fixed plant, jet pump bypassing system (see Appendix E for more details on the system). The Indian River Inlet Project is presented because it is the most recently constructed fixed plant bypassing system in the United States and should be comparable in size to other fixed plant applications. Component and installation costs will of course vary because of geographic location, inflation, and other market fluctuations; but the component costs that follow provide a recent example of the expense associated with the construction of a bypassing system. The listing (Table C-1) is by no means

Table C-1

Costs (1989) for the Indian River Inlet Bypass Plant

Item	Cost
Contractor bid, lump sum	$1,462,000
Included in Lump Sum Bid	
Eductor assemblies and extensions (2)	60,000
Pump and engine (diesel) sets (1) Supply water (340 hp) (1) Slurry booster (330 hp)	180,000
Crawler crane (135 ton rated)	500,000
Pump house (1,120 square feet) (not including pumps, piping and instrumentation)	150,000
Nuclear density and flowmeter	30,000
Pipeline (3,000 feet) (Total price installed from pump house over bridge and on north beach including brackets on bridge; base price per foot of 12-inch high-density polyethylene pipe is approximately $18 to $20)	175,000

all inclusive, but it will provide the designer with an idea of the costs of major system components. Not included in the cost breakdown (but part of the total bid cost) are contractor profit, engineering and design costs, the beach pipeline from the pump house to the beach, and other miscellaneous items. Note that the components listed here are for a jet pump system.

C-2. Other Cost Considerations. Along with various component costs, a significant amount of the overall system cost will fall under the categories of design, administration, supervision, and contingencies. Typical rates are as follows:

a. Design--6 percent of component costs.

b. Administration and Supervision--5 percent of component costs.

c. Contingencies--20 percent of component costs.

It should be restated that these values are presented as an example, and significant regional variations should be expected.

1. Mob & Demob
   Indian River Inlet (Appendix G, page G2 above) $25,000

   Using *CWCCIS* Code #20--Permanent Operating Equipment for FY83 to FY97 (Cost Index = 330.82 and 464.31, respectively) gives an increase of 40.35%. So $25,000 is converted to $35,087.50. An additional conversion in *CWCCIS* for differences between construction in Delaware and New York (FY97 Cost Index = 521.6875 and 531.3475, respectively) increases the cost by 1.85% (say 2%). Therefore, Shinnecock estimate is $35,789.25 (say $35,800).
   Shinnecock Inlet $35,800

2. a. Operations Building
      Indian River Inlet (Appendix G, page G4)--1,290 SF facility at $50/SF $64,500

      Using *CWCCIS* Code #19--Buildings, Grounds and Utilities for FY83 to FY97 (CI = 330.82 and 464.31, respectively) gives an increase of 40.35%. A $50/SF cost becomes $70.18, and when increased by the state difference of 2% becomes $71.58/SF (say $72/SF)
      Shinnecock Inlet--1,500 SF facility at $72/SF $108,000

   b. Utilities
      Indian River Inlet (Appendix G, page G4) $7,740

      Using *CWCCIS* Code #19--Building, Grounds and Utilities (see 2a. above) increases cost by 40.35% so that $7,740 becomes $10,863.09. The state difference (2%) increases it to $11,080.35 (say $11,100).
      Shinnecock Inlet utilities $11,100

   c. Security
      Indian River Inlet (Appendix G, page G4) $5,000

      Using *CWCCIS* Code #19--Building, Grounds and Utilities (see 2a. above) increases cost by 40.35% so that $5,000 becomes $7,017.50. The state difference (2%) increases it to $7,157.85 (say $7,200).
      Shinnecock Inlet security measures $7,200

3. Mechanical Equipment
   a. Indian River Inlet water supply pump & engine (Appendix G, page G4) $22,300

      As per phone conversation with Mr. Johnny Green of Standard Gravel Company, Franklinton, LA (manufacturer of jet pumps), a *Caterpillar* Model 3408 (8 cyl), 450 HP engine costs approximately $37,000. The water pump itself costs approximately $7,000. Total cost is $44,000.
      Shinnecock Inlet water supply pump & engine $44,000

   b. Indian River Inlet slurry booster pump & engine (Appendix G, page G4) $44,200

As per phone conversation with Mr. Johnny Green of Standard Gravel Company, Franklinton, LA (manufacturer of jet pumps), a *Caterpillar* Model 3412 (12 cyl), 620 HP engine costs approximately $50,000. A gravel pump with a clutch costs approximately $48,000. Miscellaneous cost for frames, etc. $22,000. Two, 300-ft long, 10-in. diameter hoses (one carrying supply water, one carrying slurry) running between jet pump and slurry booster pump @ $110/LF cost $66,000. Total cost $186,000.
Shinnecock Inlet slurry booster pump & engine $186,000

c. Shinnecock additional booster pumps and engines for the 8,800-ft pumping distance

d. Indian River Inlet jet pump (2) (Appendix G, page G4) $60,000

As per phone conversation with Mr. Johnny Green of Standard Gravel Company, Franklinton, LA (manufacturer of jet pumps), a 2.5-in. jet pump with 6-in. mixing chamber costs $50,000. Total cost for 2 jet pumps is $100,000
Shinnecock Inlet jet pumps $100,000

e. Indian River Inlet flushing water pump & motor (Appendix G, page G4) $10,600

Using *CWCCIS* Code #13--Pumping Plant for FY83 to FY97 (CI= 330.82 and 464.31, respectively) increases the cost by 40.35% making the cost $14,877.10. The state difference (2%) increases it to $15,174.64 (say $15,200).
Shinnecock Inlet flushing water pump & motor $15,200

f. Indian River Inlet air compressor (Appendix G, page G4) $8,530

Using *CWCCIS* Code #20, Permanent Operating Equipment (see 1. above), an increase of 40.35% makes the cost $11,971.86. The state difference (2%) increases it to $12,211.30 (say $12,200).
Shinnecock Inlet air compressor $12,200

g. Shinnecock Inlet instrumentation & gauges (Appendix G, page G4) $15,000

Using *CWCCIS* Code #20, Permanent Operating Equipment (see 1. above), an increase of 40.35% makes the cost $21,052.50. The state difference (2%) increases it to $21,473.55 (say $21,500).
Shinnecock Inlet instrumentation & gages $21,500

h. Indian River Inlet crawler crane (135 tons) (Appendix G, page G7) $500,000

Using *CWCCIS* Code #20, Permanent Operating Equipment, FY89 to FY97 (CI = 383.14 and 464.31, respectively) increases the cost by 21.19% and makes the cost $605,950. The state difference (2%) increases it to $618,069 (say $620,000).
Shinnecock Inlet crawler crane $620,000

3. Indian River Inlet uses one vehicle (truck) for transportation by staff in conjunction with bypass operation.

   For Shinnecock, assume initial vehicle cost of $21,700 (from Indian River Inlet replacement cost--see Appendix I, page I2)
   Shinnecock Inlet vehicle **$21,700**

4. Electrical Equipment
   Indian River Inlet (Appendix G, page G4) $39,440

   Using *CWCCIS* Code #07, Power Plant, FY83 to FY97 (CI = 324.11 and 465.09, respectively) increases cost by 43.50% making the cost $56,596.40. The state (2%) increase makes the cost $57,728.33 (say $57,700).
   Shinnecock Inlet electrical equipment **$57,700**

5. Pipes, fittings and valves
   a. Indian River Inlet piping (3000 ft of 12-in. HDPE, fittings, elbows, installation, etc.) (Appendix G, page G7) $175,000

      Fife Pipe, Inc. in Hudson, MA lists 12-in HDPE (SDR 11) at $10.75/LF of 12-in. For Shinnecock, approximately 8,800 ft of discharge pipe is required from the fixed plant to the discharge location. **$94,600**

      To reduce the impacts from possible pipe plugging, 2 additional 800-ft lines under the inlet (1,600 ft) can be installed at $10.75/LF **$17,200**

      Butt-fusion equipment rental $675/week at approximately 2 weeks (Fife Pipe, Inc) **$1,375**

      Delivery & training of butt-fusion equipment (Fife Pipe, Inc) **$750**

      Pipe delivery charges from South Carolina, 6 trucks @ $1,350/truck (Fife Pipe, Inc.) **$8,100**

   b. Indian River Inlet fittings estimated at $11,300

      Using *CWCCIS* Code #13--Pumping Plant for FY83 to FY97 (CI= 330.82 and 464.31, respectively) increases the cost by 40.35% making the cost $15,859.55. The state difference (2%) increases it to $16,176.74 (say $16,200).
      Shinnecock Inlet pipe fittings **$16,200**

c. Indian River Inlet valves (Appendix G, page G4) $30,700

Using *CWCCIS* Code #13--Pumping Plant for FY83 to FY97 (CI= 330.82 and 464.31, respectively) increases the cost by 40.35% making the cost $43,087.45. The state difference (2%) increases it to $43,949.20 (say $44,000).
Shinnecock Inlet pipe valves $44,000

d. Indian River Inlet pipe installation (Appendix G, page G4) $188,300

Using *CWCCIS* Code #13--Pumping Plant for FY83 to FY97 (CI= 330.82 and 464.31, respectively) increases the cost by 40.35% making the cost $264,279.05. The state difference (2%) increases it to $269,564.63 (say $270,000). However, because of underwater placement at Shinnecock, NAN recommends $750,000.
Shinnecock Inlet pipe installation $750,000

6. Access Road & Parking Area
Indian River Inlet access road (approx 500 ft) and parking area costs (Appendix G, page G5)
excavation and preparation (140 cy @ $10.00/cy) $1,400
wearing course (630 cy @ $6.70/cy) $4,200
base course (630 cy @ $4.90/cy) $3,100
sub-base (630 cy @ $4.40/cy) $2,800

Using *CWCCIS* Code #08, Roads, Railroads and Bridges, FY83 to FY97 (CI = 340.86 and 496.38, respectively) increases cost by 45.63%. The respective unit costs shown above with the additional 2% state increase become $14.85/cy, $9.96/cy, $7.28/cy and $6.54/cy. At Shinnecock, the required access road length is about 2,500 ft. Therefore, increase the volumes/areas by 400% and use the aforementioned costs.
excavation and preparation (560 cy @ $14.85/cy) $8,316
wearing course (2520 cy @ $9.96/cy) $25,099.20
base course (2520 cy @ $7.28/cy) $18,345.60
sub-base (2520 cy @ $6.54/cy) $16,480.80
Total cost is $68,241.60 (say $68,200).
Shinnecock Inlet access road and parking area $68,200

7. Miscellaneous
Indian River Inlet miscellaneous costs (Appendix G, page G5) $45,000

Using *CWCCIS* Code #20, Permanent Operating Equipment (see 1. above), an increase of 40.35% makes the cost $63,157.50. The state difference (2%) increases it to $64,420.65 (say $64,400).
Shinnecock Inlet miscellaneous costs $64,400

Appendix H
Annual Operating Cost Summaries for Alternative 2: Semifixed Plant

Actual Indian River Inlet annual operating costs (provided by Mr. Robert Henry of the Delaware Department of Natural Resources and Environmental Control (DNREC)) averaged for the 5-year period from 1990 through 1995 (attached) are used as a guide for estimating the annual costs for Shinnecock Inlet. Assumptions and rationale are described below.

a. For a three-person operating crew, average annual costs are $104,200 (average of $34,733 per person). Comments from U.S. Army Engineer District, New York, have suggested that $35,000 cost per person including overhead is too low and should be approximately doubled. Therefore, Shinnecock costs for a five-person crew are $350,000.

b. Electric utilities for the operations building should be approximately equal; therefore, use $1,000.

c. Plant fuel (diesel for pumps (two) and crane) averages $8,000. At Shinnecock, assume $4,000/year/pump, so with three pumps (supply, primary booster, one extra booster), use $12,000.

d. Vehicle fuel averages $600/year at Indian River Inlet. Because of increased driving due to multiple booster pumps, use $1,200.

e. At Indian River Inlet, Contract Services, Supplies and Material (CS, S&M) averages $27,000/year. This category is for miscellaneous (catch-all) costs. Assume because of the complexity of multiple boosters and longer pumping distance that this cost is double for Shinnecock. Use $54,000.

Table H1
Indian River Inlet Sand Bypassing Plant Operating Expenses

MON-YR	SAL + OEC	ELEC	FUEL (PLANT)	FUEL (TRUCK)*	GENERAL (CS,S&M)**	TOTAL $	CY PUMPED	$/CY
Feb-90	$8,374.84	$224.72	$2,079.33	$42.71	$92.00	$10,813.60	22,584	$0.48
Mar-90	$8,374.84	$414.45	$749.99	$45.51	$515.15	$10,099.94	5,167	$1.95
Apr-90	$8,374.84	$92.89	$884.72	$40.63	$272.55	$9,665.63	15,601	$0.62
May-90	$8,374.84	$44.97	$777.00	$9.75	$166.42	$9,372.98	10,188	$0.92
Jun-90	$8,374.84	$50.85	$574.27	$48.10	$367.93	$9,415.99	7,347	$1.28
Jul-90	$8,374.84	$56.13	$534.75	$50.12	$530.21	$9,546.05	8,392	$1.14
Aug-90	$8,374.84	$55.19	$627.98	$72.96	$904.24	$10,035.21	7,760	$1.29
Sep-90	$8,374.84	$51.02	$635.60	$14.40	$2,765.07	$11,840.93	12,001	$0.99
Oct-90	$8,374.84	$54.82	$1,844.53	$85.05	$1,041.25	$11,400.49	15,706	$0.73
Nov-90	$8,374.84	$52.78	$2,015.02	$35.80	$518.92	$10,997.36	3,726	$2.95
Dec-90	$8,374.84	$61.84	$594.23	$61.20	$110.54	$9,202.65	4,259	$2.16
Jan-91	$8,374.84	$73.43	$569.13	$77.62	$649.34	$9,744.36	4,105	$2.37
Feb-91	$9,305.71	$61.52	$505.07	$68.00	$1,938.76	$11,879.06	8,066	$1.47
Mar-91	$9,305.71	$65.36	$2,216.14	$45.38	$2,416.05	$14,048.64	12,888	$1.09
Apr-91	$9,305.71	$56.97	$1,005.90	$38.25	$3,738.71	$14,145.54	11,348	$1.25
May-91	$9,305.71	$60.82	$907.20	$25.76	$371.38	$10,670.87	6,656	$1.60
Jun-91	$9,305.71	$54.81	$0.00	$51.17	$1,855.94	$11,267.63	0	---
Jul-91	$9,682.51	$57.86	$0.00	$30.62	$1,059.34	$10,830.33	220	$49.23
Aug-91	$8,915.80	$53.90	$0.00	$25.20	$2,294.89	$11,289.79	74	$152.56
Sep-91	$8,236.77	$51.73	$0.00	$46.17	$622.40	$8,957.07	6,538	$1.37
Oct-91	$8,236.77	$55.49	$1,680.57	$55.28	$300.98	$10,329.09	17,912	$0.58
Nov-91	$8,236.77	$67.33	$1,313.66	$0.00	$1,718.58	$11,336.34	11,510	$0.98
Dec-91	$8,236.77	$64.66	$513.87	$40.80	$4,776.42	$13,632.52	3,018	$4.52
Jan-92	$8,236.77	$83.59	$677.34	$56.63	$2,475.04	$11,529.37	11,092	$1.04
Feb-92	$8,236.77	$103.36	$1,143.25	$86.72	$836.71	$10,406.81	5,353	$1.94
Mar-92	$8,236.77	$88.77	$567.83	$53.68	$202.57	$9,149.62	8,152	$1.12
Apr-92	$8,236.77	$70.76	$416.68	$63.12	$2,732.68	$11,520.01	2,187	$5.27
May-92	$8,611.15	$54.87	$279.32	$57.68	$339.51	$9,342.53	7,559	$1.24
Jun-92	$8,738.70	$45.66	$553.24	$32.32	$1,017.81	$10,387.73	0	---
Jul-92	$8,738.69	$59.25	$0.00	$56.40	$1,791.10	$10,645.44	0	---
Aug-92	$8,877.56	$58.41	$0.00	$87.28	$2,139.28	$11,162.53	66	$169.13
Sep-92	$8,735.22	$51.27	$867.11	$80.18	$753.82	$10,487.60	6,747	$1.55
Oct-92	$8,940.26	$51.74	$479.57	$60.90	$1,075.04	$10,607.51	16,688	$0.64
Nov-92	$8,917.15	$69.45	$662.20	$51.52	$255.07	$9,955.39	5,984	$1.66
Dec-92	$8,816.14	$66.85	$510.21	$58.20	$1,164.84	$10,616.24	3,848	$2.76
Jan-93	$8,917.12	$73.21	$681.56	$67.80	$392.04	$10,131.73	5,334	$1.90
Feb-93	$8,334.49	$103.38	$1,018.37	$61.58	$4,819.44	$14,337.26	8,485	$1.69
Mar-93	$8,334.48	$92.48	$0.00	$42.98	$985.64	$9,455.58	12,986	$0.73
Apr-93	$8,353.72	$102.72	$1,590.65	$68.78	$531.24	$10,647.11	8,560	$1.24
May-93	$8,353.71	$53.09	$803.27	$74.48	$12,026.81	$21,311.36	11,074	$1.92

(Sheet 1 of 3)

Table H1 (Continued)

MON-YR	SAL + OEC	ELEC	FUEL (PLANT)	FUEL (TRUCK)*	GENERAL (CS,S&M)**	TOTAL $	CY PUMPED	$/CY
Jun-93	$8,726.96	$52.61	$661.79	$43.58	$1,538.39	$11,023.33	87	$126.70
Jul-93	$8,944.00	$78.82	$0.00	$52.80	$742.48	$9,818.10	0	---
Aug-93	$8,926.89	$27.37	$0.00	$41.55	$4,522.55	$13,518.36	0	---
Sep-93	$8,928.54	$17.97	$0.00	$67.28	$1,361.34	$10,375.13	0	---
Oct-93	$9,073.54	$70.54	$0.00	$59.70	$939.21	$10,142.99	0	---
Nov-93	$8,913.14	$61.25	$564.15	$45.98	$22,290.31	$31,874.83	13,086	$2.44
Dec-93	$8,913.14	$91.01	$629.21	$33.08	$5,950.79	$15,617.23	7,196	$2.17
Jan-94	$8,970.14	$111.60	$956.68	$56.63	$3,270.33	$13,365.38	3,818	$3.50
Feb-94	$9,172.11	$105.36	$1,139.96	$53.93	$392.21	$10,863.57	13,841	$0.78
Mar-94	$8,974.96	$86.18	$470.92	$80.78	$1,726.10	$11,338.94	6,492	$1.75
Apr-94	$8,832.28	$76.15	$1,100.53	$53.25	$2,815.77	$12,877.98	13,926	$0.92
May-94	$8,832.28	$63.94	$1,138.38	$42.00	$339.38	$10,415.98	8,156	$1.28
Jun-94	$8,922.44	$53.91	$395.81	$91.05	$789.48	$10,252.69	0	---
Jul-94	$9,438.54	$50.67	$0.00	$46.88	$1,777.81	$11,313.90	0	---
Aug-94	$9,438.54	$50.46	$0.00	$58.50	$6,205.02	$15,752.52	0	---
Sep-94	$9,633.34	$52.81	$555.12	$42.75	$1,130.32	$11,414.34	671	$17.01
Oct-94	$9,047.89	$52.34	$1,184.29	$34.73	$1,368.15	$11,687.40	12,799	$0.91
Nov-94	$9,548.16	$64.18	$0.00	$46.88	$1,523.16	$11,182.38	12,292	$0.91
Dec-94	$7,322.25	$54.23	$1,281.13	$36.23	$524.93	$9,218.77	4,580	$2.01
Jan-95	$7,363.51	$72.93	$1,091.55	$45.00	$2,354.95	$10,927.94	7,470	$1.46
Feb-95	$7,363.43	$117.41	$666.27	$29.25	$363.48	$8,539.84	762	$11.21
Mar-95	$7,362.71	$80.92	$868.61	$51.00	$6,155.81	$14,519.05	9,439	$1.54
Apr-95	$7,099.23	$66.17	$495.37	$42.75	$2,983.40	$10,686.92	6,072	$1.76
May-95	$7,546.15	$60.56	$1,686.67	$42.90	$839.27	$10,175.55	9,496	$1.07
Jun-95	$8,551.71	$53.37	$625.26	$44.25	$1,978.41	$11,253.00	0	---
Jul-95	$9,739.54	$59.08	$0.00	$34.95	$7,116.48	$16,950.05	0	---
Aug-95	$9,686.37	$55.27	$0.00	$42.75	$3,808.54	$13,592.93	0	---
Sep-95	$9,116.82	$59.59	$0.00	$27.23	$382.01	$9,585.65	0	---
Oct-95	$10,328.27	$51.67	$674.62	$41.40	$3,170.94	$14,266.90	6,243	$2.29
Nov-95	$11,589.02	$60.03	$1,186.61	$28.80	$4,900.03	$17,764.49	10,561	$1.68
Dec-95	$12,544.60	$96.27	$1,171.77	$28.50	$6,100.20	$19,941.34	10,708	$1.86
Jan-96	$13,171.04	$66.10	$420.08	$27.00	$747.59	$14,431.81	1,329	$10.86
Feb-96	$13,033.12	$79.61	$464.51	$41.72	$1,578.32	$15,197.28	6,223	$2.44
Mar-96	$9,103.14	$78.55	$972.34	$58.80	$4,155.67	$14,368.50	10,368	$1.39
Apr-96	$9,105.44	$104.22	$1,379.18	$62.40	$576.04	$11,227.28	9,561	$1.17
May-96	$9,112.87	$30.65	$717.79	$77.25	$17,952.72	$27,891.28	4,073	$6.85
Jun-96	$9,571.65	$30.65	$350.92	$74.63	$8,421.20	$18,449.05	0	---
Jul-96	$9,994.96	$44.67	$0.00	$68.55	$476.06	$10,584.24	0	---
Aug-96	$11,127.30	$41.27	$217.32	$45.75	$9,315.57	$20,747.21	0	---

(Sheet 2 of 3)

Table H1 (Concluded)

MON-YR	SAL + OEC	ELEC	FUEL (PLANT)	FUEL (TRUCK)*	GENERAL (CS,S&M)**	TOTAL $	CY PUMPED	$/CY
Sep-96	$11,705.39	$61.28	$774.16	$66.00	$2,132.26	$14,739.09	13,901	$1.06
Oct-96	$11,610.01	$61.28	$2,146.85	$50.25	$11,734.63	$25,603.02	24,214	$1.06
Nov-96	$11,626.11	$61.29	$1,378.29	$37.80	$657.28	$13,760.77	12,407	$1.11
Dec-96	$12,176.01	$58.16	$2,461.13	$46.50	$659.94	$15,401.74	13,572	$1.13
Jan-97	$12,347.76	$79.56	$1,102.03	$79.20	$757.07	$14,365.62	12,414	$1.16
Feb-97	$11,007.09	$131.81	$1,179.50	$59.25	$6,091.41	$18,469.06	14,588	$1.27
Mar-97	$10,801.61	$108.71	$2,144.20	$81.40	$1,927.70	$15,063.62	12,730	$1.18
Apr-97	$13,276.26	$88.28	$1,730.66	$59.10	$1,164.27	$16,318.57	18,078	$0.90
May-97	$10,630.06	$77.45	$1,302.43	$17.37	$1,212.69	$13,240.00	8,616	$1.54
Jun-97	$9,802.16	$76.02	$0.00	$18.31	$1,174.75	$11,071.24	0	---
Jul-97	$10,975.69	$105.82	$0.00	$57.16	$1,112.71	$12,251.38	0	---
Aug-97	$9,496.72	$1.84	$0.00	$66.15	$5,730.37	$15,295.08	0	---
Avg/Mo	$9,260.58	$72.03	$742.48	$50.68	$2,631.93	$12,757.68	6912	$1.85

(Sheet 3 of 3)

*70-80 percent for diesel pumps.
**Contract services, supplies and materials (catch-all).

Appendix I
Capital Replacement and Overhaul Cost Summaries for Alternative 2: Semifixed Plant

Indian River Inlet capital replacement and overhaul schedule and costs (provided by Mr. Robert Henry, Delaware Department of Natural Resources and Environmental Control (DNREC)) are shown in the accompanying table. These costs and their respective overhaul/replacement intervals are used to estimate similar costs (on an annual basis) for Shinnecock Inlet using a 7-3/8 percent interest rate and 30-year return period. The Shinnecock Inlet costs are developed following the table.

Table I1
Indian River Inlet Bypass Plant Capital Replacement Costs and Schedule

Item	Action	Interval (years)	Cost ($ 1996)
Crawler Crane			
Pins, tracks, rollers, idlers, sprockets	Rebuild	6	14.0 K
Engine	Overhaul	10	8.0 K
Caterpillar Diesel Engine (for slurry pump) (Parts & labor, no crankshaft)	Overhaul	20	19.6 K
Slurry Pump			
Runner, liners, sleeve	Replace	2	7.0 K
Shell	Replace	4	5.5 K
Caterpillar Diesel Engine (for supply pump) (Parts & labor, no crankshaft)	Overhaul	16	15.7 K

(Continued)

Table I1 (Concluded)			
Item	**Action**	**Interval (years)**	**Cost ($ 1996)**
Supply Pump			
Impeller	Replace		4.7 K
Shaft bearings, packing sleeves	Replace		1.0 K
Shell	Replace	10450	10.5K
Air Compressor			
Compressor	Replace		1.2K
Motor	Replace	1015	0.9K
Gauges			
Manual (5 @ $0.2 K ea)	Replace		1.0K
Remote (4 @ $1 K ea)	Replace	510	4.0K
Flow Instrumentation (TN density & flow meters w/remote digital display panels)	Replace	10	11.0K
Crane-Mounted VHF Density Meter (Replace w/low-power UHF system)	Replace	15	2.0K
Floor Crane w/2-ton hoist	Replace	50	5.1K
¾-ton 4WD Pickup (Diesel)	Replace	6	21.7K
Slurry gate valve	Replace	6	1.2K
All other gate valves	Replace	10	3.6K
Jet Pump			
Mixing chamber	Replace	2	2.9K
Nozzle	Replace	8	1.0K
Diffuser	Replace	6	1.4K
Balance of jet pump	Replace	6	21.7K
5' steel jet pump extensions	Replace	8	2.0K
10' steel jet pump extensions	Replace	8	2.0K
300^0 steel elbows	Replace	8	1.4K
Pipeline			
Water supply (12" HDPE)	Replace	50	5.0K
Flexible dredge hoses	Replace	5	20.0K
Slurry discharge (12" HDPE)	Replace	12	30.0K
Discharge elbow (90^0)	Replace	4	1.0K
Discharge elbows (45^0)	Replace	5	0.75K
Steel dredge pipe	Replace	6	2.0K
Pumphouse			
Roof	Replace		2.8K
Furnace	Replace	2020	3.0K

Shinnecock Inlet Replacement/Overhaul Cost Calculations

Crawler Crane

a. Rebuild pins, tracks, rollers, idlers and sprockets every 6 years at a cost of $14 K per action (as per Indian River Inlet). For Shinnecock, a total of four actions will be required in the 30-year project life:

Action 1--P	=	$14 K(P/F,$7\frac{3}{8}$,6)		
		$14 K(0.6528)	=	$9,139.20
Action #2--P	=	$14 K(P/F,$7\frac{3}{8}$,12)		
		$14 K(0.4264)	=	$5,969.60
Action #3--P	=	$14 K(P/F,$7\frac{3}{8}$,18)		
		$14 K(0.2788)	=	$3,903.20
Action #4--P	=	$14 K(P/F,$7\frac{3}{8}$,24)		
		$14 K(0.1824)	=	$2,553.60
		Present Value	=	$21,565.60
		Annual value	=	$21,565.60(A/P,$7\frac{3}{8}$,30)
			=	$21,565.60(0.0837)
			=	**$1,805.04**

b. Overhaul crane engine every 10 years at a cost of $8 K per action (as per Indian River Inlet). For Shinnecock, a total of two actions will be required in the 30-year project life:

Action 1--P	=	$8K(P/F,$7\frac{3}{8}$,10)		
		$8K(0.4915)	=	$3,932.00
Action 2--P	=	$8K(P/F,$7\frac{3}{8}$,20)		
		$8K(0.2420)	=	$1,936.00
		Present Value	=	$5,868.00
		Annual Value	=	$5,868(A/P,$7\frac{3}{8}$,30)
			=	$5,868(0.0837)
			=	**$491.15**

Caterpillar Diesel Engine (slurry booster pumps)

Overhaul pump engine every 20 years at a cost of $19.6 K per action (as per Indian River Inlet). For Shinnecock, only one action will be required in the 30-year project life:

Action 1--P	=	$19.6K(P/F,$7\frac{3}{8}$,20)		
		$19.6K(0.2420)	=	$4,743.20
		Present Value	=	$4,743.20
		Annual Value	=	$4,743.20(A/P,$7\frac{3}{8}$,30)
			=	$4,743.20(0.0837)
			=	**$397.01**

Because two booster pumps will be needed, annual value = $794.02

Slurry Booster Pumps

a. Replace runner, liners and sleeve every 2 years at a cost of $7 K per action (as per Indian River Inlet). For Shinnecock, 14 actions will be required in the 30-year project life:

Action 1--P	=	$7 K(P/F,7⅜,2)		
		$7 K(0.8674)	=	$6,071.80
Action 2--P	=	$7 K(P/F,7⅜,4)		
		$7 K(0.7487)	=	$5,240.90
Action 3--P	=	$7 K(P/F,7⅜,6)		
		$7 K(0.6528)	=	$4,569.60
Action 4--P	=	$7 K(P/F,7⅜,8)		
		$7 K(0.5664)	=	$3,964.80
Action 5--P	=	$7 K(P/F,7⅜,10)		
		$7 K(0.4915)	=	$3,440.50
Action 6--P	=	$7 K(P/F,7⅜,12)		
		$7 K(0.4264)	=	$2,984.80
Action 7--P	=	$7 K(P/F,7⅜,14)		
		$7 K(0.3701)	=	$2,590.70
Action 8--P	=	$7 K(P/F,7⅜,16)		
		$7 K(0.3212)	=	$2,248.40
Action 9--P	=	$7 K(P/F,7⅜,18)		
		$7 K(0.2788)	=	$1,951.60
Action 10--P	=	$7 K(P/F,7⅜,20)		
		$7 K(0.2420)	=	$1,694.00
Action 11--P	=	$7 K(P/F,7⅜,22)		
		$7 K(0.2101)	=	$1,470.70
Action 12--P	=	$7 K(P/F,7⅜,24)		
		$7 K(0.1824)	=	$1,276.80
Action 13--P	=	$7 K(P/F,7⅜,26)		
		$7 K(0.1583)	=	$1,108.10
Action 14--P	=	$7 K(P/F,7⅜,28)		
		$7 K(0.1375)	=	$962.50
		Present Value	=	$39,575.20
		Annual Value	=	$39,575.20(A/P,7⅜,30)
			=	$39,575.20(0.0837)
			=	**$3,312.44**

Because two booster pumps will be needed, annual value = $6,624.88

b. Replace pump shell every 4 years at a cost of $5.5 K per action (as per Indian River Inlet). For Shinnecock, seven actions will be required in the 30-year project life:

Action 1--P	=	$5.5 K(P/F,7⅜,4)		
		$5.5 K(0.7487)	=	$4,117.85
Action 2--P	=	$5.5 K(P/F,7⅜,8)		
		$5.5 K(0.5664)	=	$3,115.20
Action 3--P	=	$5.5 K(P/F,7⅜,12)		
		$5.5 K(0.4264)	=	$2,345.20
Action 4--P	=	$5.5 K(P/F,7⅜,16)		
		$5.5 K(0.3212)	=	$1,766.60
Action 5--P	=	$5.5 K(P/F,7⅜,20)		
		$5.5 K(0.2420)	=	$1,331.00
Action 6--P	=	$5.5 K(P/F,7⅜,24)		
		$5.5 K(0.1824)	=	$1,003.20
Action 7--P	=	$5.5 K(P/F,7⅜,28)		
		$5.5 K(0.1375)	=	$756.25
		Present Value	=	$14,435.30
		Annual Value	=	$14,435.30(A/P,7⅜,30)
			=	$14,435.30(0.0837)
			=	**$1,208.23**

Because two booster pumps will be needed, Annual Value = $2,416.46

Caterpillar Diesel Engine (motive water supply pump)

Overhaul pump engine every 16 years at a cost of $15.7 K per action (as per Indian River Inlet). For Shinnecock, only one action will be required in the 30-year project life:

Action 1--P	=	$15.7 K(P/F,7⅜,16)		
		$15.7K(0.3212)	=	$5,042.84
		Present Value	=	$5,042.84
		Annual Value	=	$5,042.84(A/P,7⅜,30)
			=	$5,042.84(0.0837)
			=	**$422.09**

Motive Water Supply Pump

a. Replace impeller every 10 years at a cost of $4.7 K per action (as per Indian River Inlet). For Shinnecock, two actions will be required in the 30-year project life:

Action 1--P	=	$4.7K(P/F,7⅜,10)		
		$4.7K(0.4915)	=	$2,310.05
Action 2--P	=	$4.7K(P/F,7⅜,20)		
		$4.7K(0.2420)	=	$1,137.40
		Present Value	=	$3,447.45

Annual Value	=	$3,447.45(A/P,7⅜,30)
	=	$3,447.45(0.0837)
	=	**$288.55**

b. Replace shaft bearings and packing sleeves every 4 years at a cost of $1 K per action (as per Indian River Inlet). For Shinnecock, seven actions will be required in the 30-year project life:

Action 1--P	=	$1 K(P/F,7⅜,4)		
		$1 K(0.7487)	=	$748.70
Action 2--P	=	$1 K(P/F,7⅜,8)		
		$1 K(0.5664)	=	$566.40
Action 3--P	=	$1 K(P/F,7⅜,12)		
		$1 K(0.4264)	=	$426.40
Action 4--P	=	$1 K(P/F,7⅜,16)		
		$1 K(0.3212)	=	$321.20
Action 5--P	=	$1 K(P/F,7⅜,20)		
		$1 K(0.2420)	=	$242.00
Action 6--P	=	$1 K(P/F,7⅜,24)		
		$1 K(0.1824)	=	$182.40
Action 7--P	=	$1 K(P/F,7⅜,28)		
		$1 K(0.1375)	=	$137.50
		Present Value	=	$2,624.60
		Annual Value	=	$2,624.60(A/P,7⅜,30)
			=	$2,624.60(0.0837)
			=	**$219.68**

Air Compressor

a. Replace compressor every 10 years at a cost of $1.2 K per action (as per Indian River Inlet). For Shinnecock, two actions will be required in the 30-year project life:

Action 1--P	=	$1.2 K(P/F,7⅜,10)		
		$1.2 K(0.4915)	=	$589.80
Action 2--P	=	$1.2 K(P/F,7⅜,20)		
		$1.2 K(0.2420)	=	$290.40
		Present Value	=	$880.20
		Annual Value	=	$880.20(A/P,7⅜,30)
			=	$880.20(0.0837)
			=	**$73.67**

b. Replace motor every 15 years at a cost of \$0.9 K per action (as per Indian River Inlet). For Shinnecock, one action will be required in the 30-year project life:

Action 1--P	=	\$0.9 K(P/F,7⅜,15)		
		<u>\$0.9 K(0.3448)</u>	=	<u>\$310.32</u>
		Present Value	=	\$310.32
		Annual Value	=	\$310.32(A/P,7⅜,30)
			=	\$310.32(0.0837)
			=	**\$25.97**

Gauges

a. Replace manual gauges every 5 years at a cost of \$1 K per action (as per Indian River Inlet). For Shinnecock, five actions will be required in the 30-year project life:

Action 1--P	=	\$1 K(P/F,7⅜,5)		
		\$1 K(0.7009)	=	\$700.90
Action 2--P	=	\$1 K(P/F,7⅜,10)		
		\$1 K(0.4915)	=	\$491.50
Action 3--P	=	\$1 K(P/F,7⅜,15)		
		\$1 K(0.3448)	=	\$344.80
Action 4--P	=	\$1 K(P/F,7⅜,20)		
		\$1 K(0.2420)	=	\$242.00
Action 5--P	=	\$1 K(P/F,7⅜,25)		
		<u>\$1 K(0.1699)</u>	=	<u>\$169.90</u>
		Present Value	=	\$1,949.10
		Annual Value	=	\$1,949.10(A/P,7⅜,30)
			=	\$1,949.10(0.0837)
			=	**\$163.14**

b. Replace remote gauges every 10 years at a cost of \$4 K per action (as per Indian River Inlet). For Shinnecock, two actions will be required in the 30-year project life:

Action 1--P	=	\$4 K(P/F,7⅜,10)		
		\$4 K(0.4915)	=	\$1,966.00
Action 2--P	=	\$4 K(P/F,7⅜,20)		
		<u>\$4 K(0.2420)</u>	=	<u>\$968.00</u>
		Present Value	=	\$2,934.00
		Annual Value	=	\$2,934.00(A/P,7⅜,30)
			=	\$2,934.00(0.0837)
			=	**\$245.58**

Flow Instrumentation

Replace nuclear density and flow meters every 10 years at a cost of $11 K per action (as per Indian River Inlet). For Shinnecock, two actions will be required in the 30-year project life:

Action 1--P	=	\$11 K(P/F,$7\frac{3}{8}$,10)		
		\$11 K(0.4915)	=	\$5,406.50
Action 2--P	=	\$11 K(P/F,$7\frac{3}{8}$,20)		
		\$11 K(0.2420)	=	\$2,662.00
		Present Value	=	\$8,068.50
		Annual Value	=	\$8,068.50(A/P,$7\frac{3}{8}$,30)
			=	\$8,068.50(0.0837)
			=	**\$675.33**

Crane-Mounted VHF Density Meter

Replace density meter every 15 years at a cost of $2 K per action (as per Indian River Inlet). For Shinnecock, one action will be required in the 30-year project life:

Action 1--P	=	\$2 K(P/F,$7\frac{3}{8}$,15)		
		\$2 K(0.3448)	=	\$689.60
		Present Value	=	\$689.60
		Annual Value	=	\$689.60(A/P,$7\frac{3}{8}$,30)
			=	\$689.60(0.0837)
			=	**\$57.72**

¾-ton 4WD Pickup (Diesel)

Replace pickup every 6 years at a cost of $21.7 K per action (as per Indian River Inlet). For Shinnecock, four actions will be required in the 30-year project life:

Action 1--P	=	\$21.7 K(P/F,$7\frac{3}{8}$,6)		
		\$21.7 K(0.6528)	=	\$14,165.76
Action 2--P	=	\$21.7 K(P/F,$7\frac{3}{8}$,12)		
		\$21.7 K(0.4264)	=	\$9,252.88
Action 3--P	=	\$21.7 K(P/F,$7\frac{3}{8}$,18)		
		\$21.7 K(0.2788)	=	\$6,049.96
Action 4--P	=	\$21.7 K(P/F,$7\frac{3}{8}$,24)		
		\$21.7 K(0.1824)	=	\$3,958.08
		Present Value	=	\$29,468.60
		Annual Value	=	\$29,468.60(A/P,$7\frac{3}{8}$,30)
			=	\$29,468.60(0.0837)
			=	**\$2,466.52**

Slurry Gate Valve

Replace slurry gate valve every 6 years at a cost of $1.2 K per action (as per Indian River Inlet). For Shinnecock, four actions will be required in the 30-year project life:

Action 1--P	=	$1.2 K(P/F,7⅜,6)		
		$1.2 K(0.6528)	=	$783.36
Action 2--P	=	$1.2 K(P/F,7⅜,12)		
		$1.2 K(0.4264)	=	$511.68
Action 3--P	=	$1.2 K(P/F,7⅜,18)		
		$1.2 K(0.2788)	=	$334.56
Action 4--P	=	$1.2 K(P/F,7⅜,24)		
		$1.2 K(0.1824)	=	$218.88
		Present Value	=	$1,848.48
		Annual Value	=	$1,848.48(A/P,7⅜,30)
			=	$1,848.48(0.0837)
			=	**$154.72**

All other gate valves

Replace all other gate valves every 10 years at a cost of $3.6 K per action (as per Indian River Inlet). For Shinnecock, two actions will be required in the 30-year project life:

Action 1--P	=	$3.6K(P/F,7⅜,10)		
		$3.6K(0.4915)	=	$1,769.40
Action 2--P	=	$3.6K(P/F,7⅜,20)		
		$3.6K(0.2420)	=	$871.20
		Present Value	=	$2,640.60
		Annual Value	=	$2,640.60(A/P,7⅜,30)
			=	$2,640.60(0.0837)
			=	**$221.02**

Jet pump

a. Replace mixing chamber on the jet pump every 2 years at a cost of $2.9 K per action (as per Indian River Inlet). For Shinnecock, 14 actions will be required in the 30-year project life:

Action 1--P	=	$2.9 K(P/F,7⅜,2)		
		$2.9 K(0.8674)	=	$2,515.46
Action 2--P	=	$2.9 K(P/F,7⅜,4)		
		$2.9 K(0.7487)	=	$2,171.23
Action 3--P	=	$2.9 K(P/F,7⅜,6)		
		$2.9 K(0.6528)	=	$1,893.12
Action 4--P	=	$2.9 K(P/F,7⅜,8)		
		$2.9 K(0.5664)	=	$1,642.56

Action 5--P	=	$2.9 K(P/F,7⅜,10)		
		$2.9 K(0.4915)	=	$1,425.35
Action 6--P	=	$2.9 K(P/F,7⅜,12)		
		$2.9 K(0.4264)	=	$1,236.56
Action 7--P	=	$2.9 K(P/F,7⅜,14)		
		$2.9 K(0.3701)	=	$1,073.29
Action 8--P	=	$2.9 K(P/F,7⅜,16)		
		$2.9 K(0.3212)	=	$931.48
Action 9--P	=	$2.9 K(P/F,7⅜,18)		
		$2.9 K(0.2788)	=	$808.52
Action 10--P	=	$2.9 K(P/F,7⅜,20)		
		$2.9 K(0.2420)	=	$701.80
Action 11--P	=	$2.9 K(P/F,7⅜,22)		
		$2.9 K(0.2101)	=	$609.29
Action 12--P	=	$2.9 K(P/F,7⅜,24)		
		$2.9 K(0.1824)	=	$528.96
Action 13--P	=	$2.9 K(P/F,7⅜,26)		
		$2.9 K(0.1583)	=	$459.07
Action 14--P	=	$2.9 K(P/F,7⅜,28)		
		$2.9 K(0.1375)	=	$398.75
		Present Value	=	$16,095.44
		Annual Value	=	$16.095.44(A/P,7⅜,30)
			=	$16,095.44(0.0837)
			=	**$1,347.19**

b. Replace nozzle on the jet pump every 8 years at a cost of $1 K per action (as per Indian River Inlet). For Shinnecock, three actions will be required in the 30-year project life:

Action 1--P	=	$1 K(P/F,7⅜,8)		
		$1 K(0.5664)	=	$566.40
Action 2--P	=	$1 K(P/F,7⅜,16)		
		$1 K(0.3212)	=	$321.20
Action 3--P	=	$1 K(P/F,7⅜,24)		
		$1 K(0.1824)	=	$182.40
		Present Value	=	$1,070.00
		Annual Value	=	$1,070.00(A/P,7⅜,30)
			=	$1,070.00(0.0837)
			=	**$89.56**

c. Replace diffuser on the jet pump every 6 years at a cost of $1.4 K per action (as per Indian River Inlet). For Shinnecock, four actions will be required in the 30-year project life:

Action 1--P	=	$1.4 K(P/F,$7\frac{3}{8}$,6)		
		$1.4 K(0.6528)	=	$913.92
Action 2--P	=	$1.4 K(P/F,$7\frac{3}{8}$,12)		
		$1.4 K(0.4264)	=	$596.96
Action 3--P	=	$1.4 K(P/F,$7\frac{3}{8}$,18)		
		$1.4 K(0.2788)	=	$390.32
Action 4--P	=	$1.4 K(P/F,$7\frac{3}{8}$,24)		
		$1.4 K(0.1824)	=	$255.36
		Present Value	=	$2,156.56
		Annual Value	=	$2,156.56(A/P,$7\frac{3}{8}$,30)
			=	$2,156.56(0.0837)
			=	**$180.50**

d. Replace balance of jet pump every 6 years at a cost of $21.7 K per action (as per Indian River Inlet). For Shinnecock, four actions will be required in the 30-year project life:

Action 1--P	=	$21.7 K(P/F,$7\frac{3}{8}$,6)		
		$21.7 K(0.6528)	=	$14,165.76
Action 2--P	=	$21.7 K(P/F,$7\frac{3}{8}$,12)		
		$21.7 K(0.4264)	=	$9,252.88
Action 3--P	=	$21.7 K(P/F,$7\frac{3}{8}$,18)		
		$21.7 K(0.2788)	=	$6,049.96
Action 4--P	=	$21.7 K(P/F,$7\frac{3}{8}$,24)		
		$21.7 K(0.1824)	=	$3,958.08
		Present Value	=	$33,426.68
		Annual Value	=	$33,426.68(A/P,$7\frac{3}{8}$,30)
			=	$33,426.68(0.0837)
			=	**$2,797.81**

e. Replace 5-ft steel jet pump extensions every 8 years at a cost of $2 K per action (as per Indian River Inlet). For Shinnecock, three actions will be required in the 30-year project life:

Action 1--P	=	$2 K(P/F,$7\frac{3}{8}$,8)		
		$2 K(0.5664)	=	$1,132.80
Action 2--P	=	$2 K(P/F,$7\frac{3}{8}$,16)		
		$2 K(0.3212)	=	$642.40
Action 3--P	=	$2 K(P/F,$7\frac{3}{8}$,24)		
		$2 K(0.1824)	=	$364.80
		Present Value	=	$2,140.00

Annual Value	=	$2,140.00(A/P,7⅜,30)
	=	$2,140.00(0.0837)
	=	**$179.12**

f. Replace 10-ft steel jet pump extensions every 8 years at a cost of $2 K per action (as per Indian River Inlet). For Shinnecock, three actions will be required in the 30-year project life:

Action 1--P	=	$2 K(P/F,7⅜,8)		
		$2 K(0.5664)	=	$1,132.80
Action 2--P	=	$2 K(P/F,7⅜,16)		
		$2 K(0.3212)	=	$642.40
Action 3--P	=	$2 K(P/F,7⅜,24)		
		$2 K(0.1824)	=	$364.80
		Present Value	=	$2,140.00
		Annual Value	=	$2,140.00(A/P,7⅜,30)
			=	$2,140.00(0.0837)
			=	**$179.12**

g. Replace 30-deg steel elbows every 8 years at a cost of $1.4 K per action (as per Indian River Inlet). For Shinnecock, three actions will be required in the 30-year project life:

Action 1--P	=	$1.4 K(P/F,7⅜,8)		
		$1.4 K(0.5664)	=	$792.96
Action 2--P	=	$1.4 K(P/F,7⅜,16)		
		$1.4 K(0.3212)	=	$449.68
Action 3--P	=	$1.4 K(P/F,7⅜,24)		
		$1.4 K(0.1824)	=	$255.36
		Present Value	=	$1,498.00
		Annual Value	=	$1,498.00(A/P,7⅜,30)
			=	$1,498.00(0.0837)
			=	**$125.38**

Pipeline

a. Replace flexible dredge hoses every 5 years at a cost of $20 K per action (as per Indian River Inlet). For Shinnecock, five actions will be required in the 30-year project life:

Action 1--P	=	$20 K(P/F,7⅜,5)		
		$20 K(0.7009)	=	$14,018.00
Action 2--P	=	$20 K(P/F,7⅜,10)		
		$20 K(0.4915)	=	$9,830.00
Action 3--P	=	$20 K(P/F,7⅜,15)		
		$20 K(0.3448)	=	$6,896.00

Action 4--P	=	\$20 K(P/F,$7\frac{3}{8}$,20)		
		\$20 K(0.2420)	=	\$4,840.00
Action 5--P	=	\$20 K(P/F,$7\frac{3}{8}$,25)		
		\$20 K(0.1699)	=	\$3,398.00
		Present Value	=	\$38,982.00
		Annual Value	=	\$38,982.00(A/P,$7\frac{3}{8}$,30)
			=	\$38,982.00(0.0837)
			=	**\$3,262.79**

b. Replace 12-in. HDPE slurry discharge pipeline every 12 years at a cost of \$10.75/LF (as per Fife Pipe, Inc.). Total replacement cost for 8,000 ft of pipeline (8,800 ft less 800 ft under the inlet) is \$86 K per action. For Shinnecock, two actions will be required in the 30-year project life:

Action #1--P	=	\$86 K(P/F,$7\frac{3}{8}$,12)		
		\$86 K(0.4264)	=	\$36,670.40
Action 2--P	=	\$86 K(P/F,$7\frac{3}{8}$,24)		
		\$86 K(0.1824)	=	\$15,686.40
		Present Value	=	\$52,356.80
		Annual Value	=	\$52,356.80(A/P,$7\frac{3}{8}$,30)
			=	\$52,356.80(0.0837)
			=	**\$4,382.26**

c. Replace discharge elbow (90 deg) every 4 years at a cost of \$1 K per action (as per Indian River Inlet). For Shinnecock, a total of seven actions will be required for the 30-year project life:

Action 1--P	=	\$1 K(P/F,$7\frac{3}{8}$,4)		
		\$1 K(0.7487)	=	\$748.70
Action 2--P	=	\$1 K(P/F,$7\frac{3}{8}$,8)		
		\$1 K(0.5664)	=	\$566.40
Action 3--P	=	\$1 K(P/F,$7\frac{3}{8}$,12)		
		\$1 K(0.4264)	=	\$426.40
Action 4--P	=	\$1 K(P/F,$7\frac{3}{8}$,16)		
		\$1 K(0.3212)	=	\$321.20
Action 5--P	=	\$1 K(P/F,$7\frac{3}{8}$,20)		
		\$1 K(0.2420)	=	\$242.00
Action 6--P	=	\$1 K(P/F,$7\frac{3}{8}$,24)		
		\$1 K(0.1824)	=	\$182.40
Action 7--P	=	\$1 K(P/F,$7\frac{3}{8}$,28)		
		\$1 K(0.1375)	=	\$137.50
		Present Value	=	\$2,624.60

Annual Value	=	\$2,624.60(A/P,$7\frac{3}{8}$,30)
	=	\$2,624.60(0.0837)
	=	**\$219.67**

d. Replace discharge elbows (45 deg) every 5 years at a cost of \$0.75 K per action (as per Indian River Inlet). For Shinnecock, a total of five actions will be required for the 30-year project life:

Action 1--P	=	\$0.75 K(P/F,$7\frac{3}{8}$,5)		
		\$0.75 K(0.7009)	=	\$525.68
Action 2--P	=	\$0.75 K(P/F,$7\frac{3}{8}$,10)		
		\$0.75 K(0.4915)	=	\$368.63
Action 3--P	=	\$0.75 K(P/F,$7\frac{3}{8}$,15)		
		\$0.75 K(0.3448)	=	\$258.60
Action 4--P	=	\$0.75 K(P/F,$7\frac{3}{8}$,20)		
		\$0.75 K(0.2420)	=	\$181.50
Action 5--P	=	\$0.75 K(P/F,$7\frac{3}{8}$,25)		
		\$0.75 K(0.1699)	=	\$127.43
		Present Value	=	\$1,461.84
		Annual Value	=	\$1,461.84(A/P,$7\frac{3}{8}$,30)
			=	\$1,461.84(0.0837)
			=	**\$122.36**

e. Replace steel dredge pipe every 6 years at a cost of \$2 K per action (as per Indian River Inlet). For Shinnecock, four actions will be required in the 30-year project life:

Action 1--P	=	\$2 K(P/F,$7\frac{3}{8}$,6)		
		\$2 K(0.6528)	=	\$1,305.60
Action 2--P	=	\$2 K(P/F,$7\frac{3}{8}$,12)		
		\$2 K(0.4264)	=	\$852.80
Action 3--P	=	\$2 K(P/F,$7\frac{3}{8}$,18)		
		\$2 K(0.2788)	=	\$557.60
Action 4--P	=	\$2 K(P/F,$7\frac{3}{8}$,24)		
		\$2 K(0.1824)	=	\$364.80
		Present Value	=	\$3,080.80
		Annual Value	=	\$3,080.80(A/P,$7\frac{3}{8}$,30)
			=	\$3,080.80(0.0837)
			=	**\$257.86**

Pumphouse

a. Replace roof of pumphouse every 20 years at a cost of \$2.8 K per action (as per Indian River Inlet). For Shinnecock, a total of one action will be required in the 30-year project life:

Action 1--P	=	\$2.8 K(P/F,$7\frac{3}{8}$,20)		
		<u>\$2.8 K(0.2420)</u>	=	<u>\$677.60</u>
		Present Value	=	\$677.60
		Annual Value	=	\$677.60(A/P,$7\frac{3}{8}$,30)
			=	\$677.60(0.0837)
			=	**\$56.72**

b. Replace furnace of pumphouse every 20 years at a cost of \$3 K per action (as per Indian River Inlet). For Shinnecock, a total of one action will be required in the 30-year project life:

Action 1--P	=	\$3 K(P/F,$7\frac{3}{8}$,20)		
		<u>\$3 K(0.2420)</u>	=	<u>\$726.00</u>
		Present Value	=	\$726.00
		Annual Value	=	\$726.00(A/P,$7\frac{3}{8}$,30)
			=	\$726.00(0.0837)
			=	**\$60.77**

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Gregory L. Williams, Andrew Morang, Linda Lillycrop

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13. ABSTRACT *(Maximum 200 words)*

The U.S. Army Engineer District, New York, is conducting several shore protection studies along the south shore of Long Island, New York. Shinnecock Inlet is the easternmost of six openings in the barrier island chain that runs along the south shore of Long Island and the inlet falls within the largest shore protection effort, the "Fire Island to Montauk Point Reformulation Study (FIMPRS)." Under FIMPRS, coastal processes, shore protection, and flood damage reduction alternatives are being examined from Fire Island Inlet eastward to Montauk Point. This report discusses the geologic history of the inlet and using the results of a coastal processes study, evaluates ebb shoal morphology and longshore transport processes as they relate to sand management (bypass) options. Five bypass options are evaluated based on cost, operational effectiveness, and sand source location. A decision matrix is included to compare alternatives.

14. SUBJECT TERMS

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Ebb shoal morphology	Sand bypassing	Shore Protection
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